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ENGINEERING AND DESIGN

STABILITY ANALYSIS OF CONCRETE STRUCTURES

ENGINEER MANUAL

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DEPARTMENT OF THE ARMY
U.S. Army Corps of Engineers
Washington, D.C. 20314-1000

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No. 1110-2-2100

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1. Purpose. This manual provides guidance for stability analysis of concrete gravity structures. Stability refers to resistance to sliding and floatation, limits on the eccentricity of the resultant of the applied loads, and limits on the bearing capacity of the foundation materials. This manual applies to all types of structures founded on rock or soil, such as: dams, outlet works, navigation locks, floodwalls, and pumping stations. This manual is not applicable to piles or caissons, or to structures founded on these elements.

2. Applicability. This manual applies to all USACE commands having responsibilities for Civil Works projects.

3. Distribution. This manual is approved for public release. Distribution is unlimited.

4. Discussion. This manual is written to be compatible with risk-based planning and design methods currently being implemented within USACE. It consolidates and standardizes stability requirements, which were previously contained in several other publications. Changes contained in Chapters 3 and 4 will provide adequate safety factors for all types of structures and loading conditions, while reducing excess conservatism for infrequent loadings of short duration. This will result in project cost savings when compared to some structures designed using previous criteria. Stability criteria in other manuals is being revised to be consistent with this manual. In the interim, where there are conflicting stability criteria, the provisions of this manual shall govern.

FOR THE COMMANDER:

JOHN R. MCMAHON
Colonel, Corps of Engineers
Chief of Staff

This manual supersedes EC 1110-2-6058 dated 30 November 2003

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Chapter 1
General

1-1. Purpose

This manual establishes and standardizes stability criteria for use in the design and evaluation of the many various types of concrete structures common to Corps of Engineers civil works projects. As used in this manual, the term “stability” applies to external global stability (sliding, rotation, flotation and bearing), not to internal stability failures such as sliding on lift surfaces or exceedance of allowable material strengths.

1-2. Applicability

This manual applies to all USACE commands having responsibilities for civil works projects.

1-3. References

Required and related publications are listed in Appendix A.

1-4. Distribution Statement

Approved for public release, distribution is unlimited.

1-5. Mandatory Requirements

Designers performing stability analyses of concrete structures are required to satisfy specific mandatory requirements. The purpose of mandatory requirements is to assure the structure meets minimum safety and performance objectives. Mandatory requirements usually pertain to critical elements of the safety analysis such as loads, load combinations and factors of safety. Mandatory requirements pertaining to the guidance contained in a particular chapter are summarized at the end of that chapter. No mandatory requirements are identified in the appendices. Instead, any mandatory requirements pertaining to information contained in appendices is cited in chapters which refer to those appendices. Where other Corps guidance documents are referenced, the designer must review each document to determine which of its mandatory requirements are applicable to the stability analysis. Engineers performing the independent technical review must ensure that the designers have satisfied all mandatory requirements. Waiver procedures for mandatory requirements are described in ER 1110-2-1150. This reference also indicates that deviation from non-mandatory provisions should be rare, and are subject to approval by the engineering chief in the design district.

1-6. Scope

This manual covers requirements for static methods used in stability analyses of hydraulic structures. The types of concrete structures addressed in this manual include dams, locks, retaining walls, inland floodwalls, coastal floodwalls, spillways, outlet works, hydroelectric power plants, pumping plants, and U-channels. The structures may be founded on rock or soil and have either flat or sloped bases. Pile-founded structures, sheet-pile structures, and footings for buildings are not included. When the stability requirements of this manual conflict with those in other Engineering Manuals or Engineering Technical Letters, the requirements of this manual shall govern. These requirements apply to all potential failure planes at or slightly below the structure/foundation interface. They also apply to certain potential failure planes within unreinforced concrete gravity structures. This manual defines the types and combination of applied loads, including uplift forces due to hydrostatic pressures in the foundation material. The manual defines the various components that enable the structure to resist movement, including anchors to the foundation. Most importantly, the manual prescribes the safety factors, which govern stability requirements for the structure for various load combinations. Also, guidance is provided for evaluating and improving the stability of existing structures.

1-7. Background

a. General. Engineer Manuals published over the past 40 years have set stability requirements for the different major civil works structures. For sliding and bearing, the stability requirements have been expressed deterministically in terms of an explicit factor of safety that sets the minimum acceptable ratio of foundation strength along the most critical failure plane to the design loads applied to the failure plane. The analysis for determination of the resultant location in prior guidance has been termed an *overturning stability analysis*. This is a misnomer since a foundation bearing, crushing of the structure toe, and/or a sliding failure will occur before the structure overturns. This manual replaces the term overturning stability analysis with resultant location.

b. Intent. The basic intent of the new guidance specified herein is summarized below:

- Provide new standard factors of safety as replacement for the somewhat variable factors of safety previously specified in other Corps guidance documents.

- Establish basic structural performance goals for each loading condition category.

- Provide tabular summaries of the structure-specific loading-condition check lists found in the other Corps guidance documents in order to properly categorize each loading condition as either usual, unusual, or extreme.

- Require the use of higher factors of safety for conditions where site information is not sufficient to provide a high degree of confidence with respect to the reliability of foundation strength parameters, loads information, and analytical procedures used in the stability analysis.

- Permit the use of lower factors of safety for existing structures when there is a high degree of confidence, based on records of construction and in-service conditions, that the values of the critical parameters used in the stability analysis are accurate.

The process used to standardize factors of safety is based on the premise that the traditional factors of safety specified in the recent guidance for Corps concrete hydraulic structures, for the most part, provide adequate protection against stability failure. The standardization process recognizes, as did previous Corps guidance, that lower factors of safety can be assigned to those loads and loading conditions designated as *unusual*, or *extreme* because the probabilities of those loads and load conditions occurring during the life of the structure are significantly less than the probabilities for *usual* loading conditions. The following elements were part of the safety factor standardization process:

- Traditional factors of safety specified in previous Corps guidance documents were used as a basis for establishing new factors of safety, which are re-formatted to be consistent with other Corps guidance that has probabilistic based requirements.

- The guidance incorporates past practices of assigning lower factors of safety to *normal* structures, as compared to those traditionally used for *critical* structures,.

- The guidance incorporates past practices of categorizing maintenance and construction loads as *unusual* loads.

- The guidance defines the loading condition categories of *usual*, *unusual*, and *extreme* in probabilistic terms to provide standardization as to which category various structure specific loadings should be assigned.

- Provides a consistent set of safety factors, which account for loading probability, critical structures, and the knowledge of site information used in the stability analysis.

c. Factors of safety. Factors of safety are needed in stability and structural analyses because of the potential variability in loads and material strengths. The factor of safety assigned to a particular stability design or investigation reduces the risk of unsatisfactory performance due to loads being greater than assumed for design and the risk of unsatisfactory performance due to material strengths being less than assumed for design. This guidance makes no attempt to quantify the reliability of the safety factors prescribed for use in the design and evaluation of Corps structures other than that they are traditionally accepted values that when used with prescribed simple assessment procedures have produced structures which have performed satisfactorily for many years. The minimum-allowable safety factors described in this manual assume that a complete and comprehensive geotechnical investigation has been performed. Higher safety factors are required when site information is limited. When concerns about stability exist, the designer should take all measures necessary to quantify load and material strength variability and use the most comprehensive analytical tools available to evaluate the capacity of the structure to meet performance objectives.

d. Sliding stability. Sliding of a structure on its foundation represents the most difficult aspect of a stability analysis, especially in those instances where the foundation is jointed, and where the strength properties vary throughout the foundation. The approach to evaluating sliding stability is one that uses the limit equilibrium method with the linear Mohr-Coulomb failure criterion as a basis for estimating maximum available shear strength. The greatest uncertainties in the analysis are those associated with shear strength determination. The safety factors used are consistent with current foundations and explorations procedures, and with current analytical methods. The guidance recognizes that there are foundations where design shear strength parameters are highly variable because foundation conditions change from one area of the foundation to another and because the foundation may be comprised of both intact rock, and jointed rock with clean or filled discontinuities all with differing shear / displacement characteristics and possibly with strain-softening characteristics which make overall strength a function of displacement. A combination of experience and judgment is necessary to confidently determine that the strength and load parameters used in the stability analysis will provide structures that meet performance objectives.

1-8. Coordination

Even though stability analysis of concrete structures is a structural engineering responsibility, the analysis must be performed with input from other disciplines. It is necessary to determine hydrostatic loads consistent with water levels determined by hydraulic and hydrological engineers. Geotechnical engineers and geologists must provide information on properties of foundation materials, and must use experience and judgement to predict behavior of complex foundation conditions. To ensure that the proper information is supplied, it is important that those supplying the information understand how it will be used by the structural engineer. To ensure that the information is applied appropriately, it is important that the structural engineer understand methods and assumptions used to develop this interdisciplinary data.

Chapter 2
Failure Modes and Wedge Sliding Analysis

2-1. General

The objective of a stability analysis is to maintain horizontal, vertical, and rotational equilibrium of the structure. Geotechnical information is needed to properly define and perform a realistic stability analysis. Possible failure modes and planes of weakness must be determined from onsite conditions, material strengths, and uplift forces. Stability is ensured by:

- Providing an adequate factor of safety against sliding at all possible failure planes.
- Providing specific limitations on the magnitude of the foundation bearing pressure.
- Providing constraints on the permissible location of the resultant force on any plane.
- Providing an adequate factor of safety against flotation of the structure.

However, satisfying the above provisions may not ensure stability if the structure experiences significant loss of foundation material due to erosion or piping, or if there is an internal failure due to inadequate strength of the structural materials. Stability is just one of the requirements necessary to ensure adequate structural performance.

2-2. Limit Equilibrium Analysis

The forces and pressures acting on a structure are indeterminate. Static equilibrium equations are insufficient to obtain a solution for lateral soil forces; additional assumptions must be incorporated in the analysis. For nonlinear materials, such as soils, this is commonly done by assuming that a limit or failure state exists along some surface and that the shear force along the surface corresponds to the shear strength of the material. With these assumptions, equilibrium equations can be solved. Hence, this approach is commonly called limit-equilibrium analysis. To ensure that the assumed failure does not occur, a reduction factor (safety factor or strength mobilization factor) is applied to the material strength. It should be noted that this approach differs significantly from that commonly used for indeterminate structure analysis, where stress-strain properties and deformations are employed. This limit equilibrium approach provides no direct information regarding deformations; it is implied that deformations are sufficient to induce the failure condition. Actual deformations will vary non-linearly in response to actual applied loads. Deformations are indirectly limited to tolerable values by the judicious selection of a safety factor.

2-3. Sliding Planes

Stability must be assessed on selected surfaces within structure in accordance with the methods presented in EM 1110-2-2200. Sliding safety must also be assessed at/or near the foundation-structure interface. This surface may be either level or sloping. Generally, it may be assumed that a surface that slopes upward (in the direction of possible sliding) will have a beneficial effect, while one that slopes downward will increase the possibility for sliding. Figure 2-1 illustrates the beneficial and adverse effects of base slope. Where a shallow weak seam exists below a structure's contact with the foundation, or a structure is imbedded below the top of the foundation, two possible failure modes are present. One mode involves slippage along the weak plane (directly under the structure) and along its extension until it daylights. The other mode involves slippage along the weak plane directly under the structure plus slippage along a plane through the foundation above the weak seam (crossbed shear for rock or passive resistance for soil). When the weak seam extends a large distance past the toe of the structure without daylighting, the second mode will usually be critical. Figure 2-2 illustrates these modes of failure.

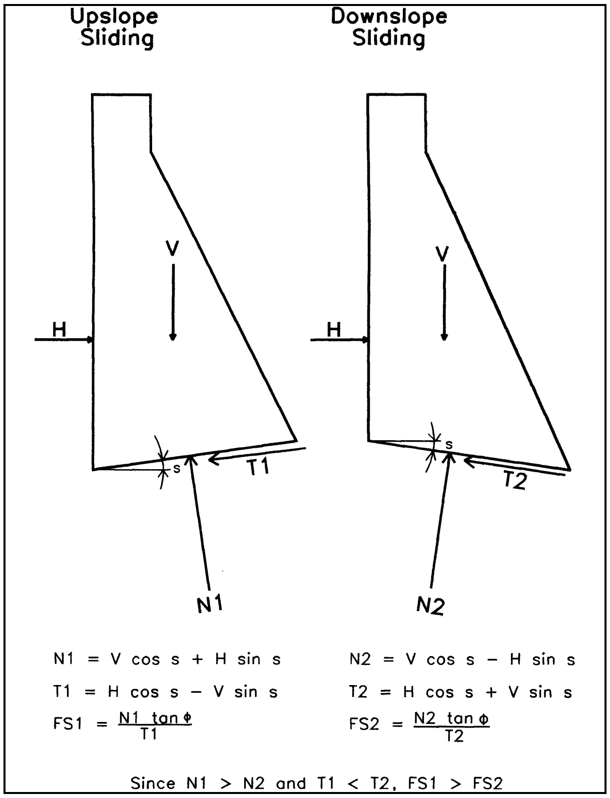


Figure 2-1

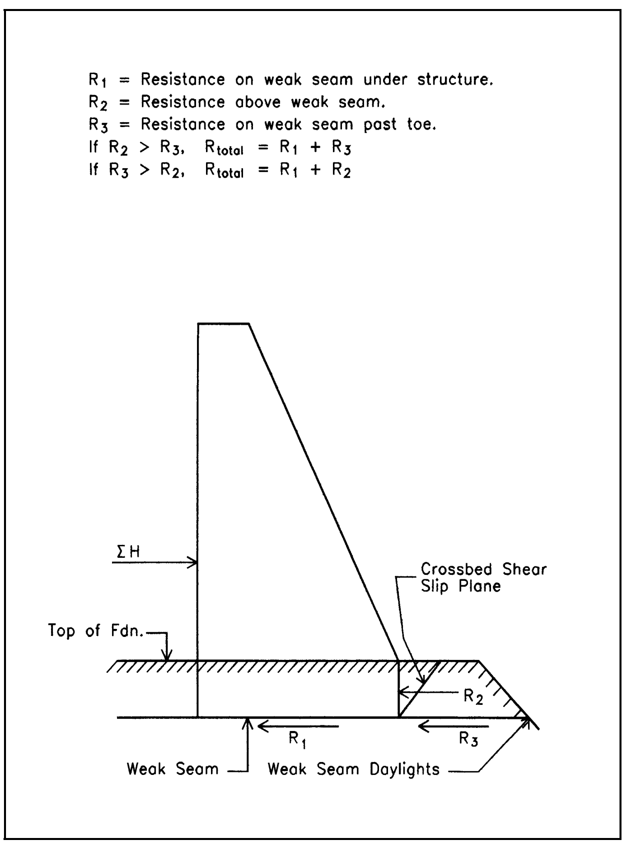


Figure 2-2

2-4. Resultant Location

Conformance with resultant location requirements ensures that the structure is safe from rotational failure. The slope of the resultant and its location are critical in assessing the foundation bearing capacity. For some load condition categories, the resultant is allowed to fall outside the middle-third of the base. In these instances, it is assumed that the structure-foundation interface has no capability for resisting tensile stresses; therefore, part of the structure's base is assumed to lose contact with the foundation resulting in changes to the uplift pressure acting on the base.

2-5. Flotation

This mode of failure occurs when the net uplift force (gross uplift on the base minus the weight of surcharge water above the structure) exceeds the summation of forces due to the weight of the structure, the weight of water contained in the structure, and other surcharge loads.

2.6. Bearing

Analytical methods, traditional bearing capacity equations, field tests, and laboratory tests are all used to determine the bearing capacity of soil and rock. The allowable bearing capacity is defined as the maximum pressure that can be permitted on a foundation soil or rock mass giving consideration to all pertinent factors, with adequate safety against rupture of the soil or rock mass, or settlement of the foundation of such magnitude as to jeopardize the performance and safety of the structure. Increases in allowable bearing capacity are permitted for unusual and extreme load conditions over those required for usual load conditions. The allowable increases are provided in Chapter 3. Shear strength parameters used in the determination of bearing capacity values shall be established in accordance with the discussion presented in Paragraph 2-8.

a. *Soil.* For structures founded on soil, the bearing capacity is limited by the ability of the soil to safely carry the pressure placed on the soil from the structure without undergoing a shear failure. Prevention of a shear failure, however, does not ensure that settlements will be within acceptable limits, therefore, a settlement analysis is usually performed in addition to the shear analysis. Discussion on methods for estimating settlements and limitations in accuracy of settlement analyses is contained in EM 1110-1-1904. Methods for determining allowable bearing capacity of soils are covered in EM 1110-1-1905. General shear failure in a homogeneous soil foundation, for a vertical loading applied at the middle of a structure with a horizontal base-foundation contact, is illustrated by Prandtl's arc of shear failure as shown in Figure 2-3. The shape of the failure surface will be affected by eccentricity, the presence of shear components, and slope of the contact surface. Eccentricity and the presence of shear components will tend to make this type of failure more probable, while a sloping contact surface can either increase or decrease the probability of failure depending upon the slope direction.

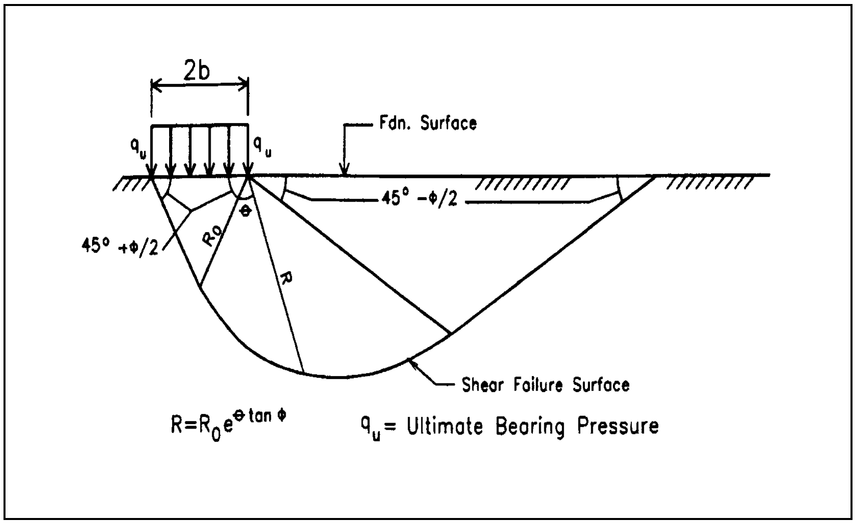


Figure 2-3

b. *Rock.* For structures founded on rock, failure modes may consist of local crushing, shear failures on weak seams, and failures at discontinuities or along bedding planes. The bearing capacity of rock will depend on whether the rock is intact, jointed, layered, or fractured. Methods used for determination of rock bearing capacities are contained in EM 1110-1-2908. The design for structures on rock foundations will involve sliding stability analyses as well as bearing capacity and settlement analyses. Sliding stability analyses address the ability of the rock foundation to resist the imposed loads without the occurrence of shearing on any horizontal or sloping weak plane. Basic rock foundation data that should be obtained for use during the design stage include material properties, strike, dip, thickness, and discontinuities such as faults, fissures and fractures. Such information should be incorporated into the bearing capacity, settlement, and sliding stability analyses.

c. *Coordination between disciplines.* The structural engineer and geotechnical engineer/geologist must coordinate their efforts in order to properly evaluate the bearing capacity of a foundation. Bearing capacity is affected by the size and shape of structure's base, the type of structure, type of loading (static or dynamic), load duration, the eccentricity of the load acting on the foundation, and the shear components of the load; all of which should be furnished to the geotechnical engineer/geologist by the structural engineer. The location and identification of weak zones and planes or discontinuities, soil and rock strength parameters, information on existing faults, and the allowable bearing capacity of the foundation should be furnished to the structural engineer by the geotechnical engineer/geologist.

2-7. Geotechnical Explorations and Testing

The scope of any geotechnical investigation will depend on geological structural complexity, imposed or existing loads acting on the foundation, and to some extent the consequences should a failure occur. The complexity of the foundation will determine the extent of drill holes, mapping, trenching, and other exploratory measures, which may be required to accurately describe foundation conditions. Guidelines for foundation explorations and testing are provided in EM 1110-1-1802, EM 1110-1-1804, and EM 1110-1-2908.

2-8. Shear Strength Tests

Shear strength parameters required for bearing capacity and sliding stability analyses may be estimated for soils from the results of in situ tests and/or by direct shear and triaxial tests performed in the laboratory. For rock, these values are usually obtained from laboratory tests. Shear strength is a function of many complex independent variables including mineralogy; particle size, shape and gradation; cementation; degree of consolidation; state of stress; anisotropy; and drainage conditions. Therefore, any tests performed in the laboratory should model the conditions that will occur during project operation. Since shearing may take place on any plane that includes intact rock, sheared rock, or jointed rock, strength values for all differing rock conditions must be established in order that sliding stability and bearing capacity may be determined. EM 1110-2-1906 provides guidance on laboratory soils testing. Procedures for testing soils are also described in EM 1110-1-1804 and EM 1110-2-1913. Procedures for testing rock specimens are given in EM 1110-1-2908 and the "Rock Testing Handbook" (U. S. Army Waterways Experiment Station 1980). Coordination must be maintained between the structural engineer and the geotechnical engineer/geologist to ensure that safe and economical designs are obtained.

a. Soils Tests.

(1) *In-situ soils tests.*

- Standard Penetration Test (SPT). The SPT resistance, often referred to as the blow count, is frequently used to estimate the relative density of soil. This relative density can then be correlated with the angle of internal friction, ϕ, and the undrained shear strength, c.

- Cone Penetration Test (CPT). The CPT may also be used to estimate the relative density of cohesionless soils and the undrained shear strength of cohesive soils. The CPT is especially suitable for sands, where it is preferable to the SPT.

- Field Vane Test (FVT). The FVT is commonly used to estimate the in situ undrained cohesive strength of soft to firm clays.

(2) Laboratory soils tests.

- Q-Test. In a Q test the water content of the soil sample is not permitted to change either prior to or during load application. The Q test produces results that approximate the shear strength available for short-term loading conditions. In cohesive soils this test yields relatively large c (cohesion) values and very low or zero ϕ values.

- R-Test. The R test represents conditions in which impervious or semi-pervious soils that have been consolidated under one set of stresses are subjected to a stress change without time for further change in water content prior to failure. In cohesive soil this test furnishes undrained shear strength parameters.

- S-Test. An S test is used to measure drained or effective stress strength parameters, c' and ϕ'. The soil sample is consolidated under an initial confining stress and loading increments are applied slowly enough to allow pore water pressures to dissipate with each load increment during shear. Results of S tests are applicable to free-draining soils where excess pore pressures do not develop during shear. S tests are also used for evaluating the shear strength of cohesive soils under long term loading conditions, where excess pore pressures have dissipated.

b. Rock Tests. The tests used for evaluating rock shear strength parameters should mirror, as closely as possible, the conditions that are expected to exist in the field. The structural engineer is referred to EM 1110-1-2908, and numerous references contained therein, in order to become familiar with such things as rock descriptors, rock mass classification systems, laboratory classification and index tests for rock, selection of modulus of deformation, etc.

(1) In-situ rock tests.

- In-Situ Direct Shear Test. In-situ direct shear tests are expensive and are only performed where critically located, thin, weak, continuous seams exist within relatively strong adjacent rock. Relatively large surface areas must be tested in order to address unknown scale effects. The test, as performed on thin, fine grained, clay seams, is considered to be an undrained test.

- In-Situ Uniaxial Compression Test. In-situ uniaxial compressive tests are expensive. This test is used to measure the elastic properties and compressive strength of large volumes of virtually intact rock in an unconfined state. The results obtained are useful in evaluating the effects of scale, however, the test is seldom used just for this purpose.

(2) Laboratory rock tests.

- Unconfined Uniaxial Compression Test. This test is performed primarily to obtain the unconfined compression strength and the elastic properties of a rock sample. Poisson's ratio can be determined if longitudinal and lateral strain measurements are taken during the test. Occasionally samples are tested with differing orientations in order to describe three-dimensional anisotropy. This test may not be indicative of the overall rock mass strength.

- Triaxial Test. The triaxial test can be made on intact, cylindrical rock samples. Data is provided for determination of rock strength in the drained or undrained state when subjected to three-dimensional loading. Data from the test, used in calculations, provides the strength and elastic properties of the sample at various confining pressures. Strengths along planes of weakness, such as natural joints, seams, and bedding, can be determined if these planes are properly oriented in the test. The oriented plane variation is particularly useful for strengths on thinly filled discontinuities containing soft materials. Since clean discontinuities are free draining, tests performed on them should be drained tests. Tests on discontinuities filled with coarse-grained materials should also be drained tests. The tests for discontinuities filled with fine-grained materials should be undrained tests.

- Laboratory Direct Shear Test. The laboratory direct shear test is primarily used to measure the shear strength, at various normal stresses, along planes of discontinuity or weakness. When this test is performed on the surface of a clean discontinuity (with asperities) that is subjected to very high normal stress, with a rapidly applied shear stress and small deformations, the values obtained will represent the *peak shear strength*. The test is often performed at a reduced rate of shear stress application, with intermediate or low normal stresses, and with the asperities over-ridden, resulting in reduced values for shear strength. Repetitive shearing of a sample, or continuing displacement to a point where shear strength is constant, establishes the *residual shear strength* that is available. For the test results to be valid, test conditions must be as close as possible to the conditions that will exist in the field. Test drainage conditions should be essentially the same as for the triaxial tests discussed above. *Upper bound* and *lower bound* shear strengths are discussed in EM 1110-1-2908.

2-9. Selection of Design Shear Strengths

Design shear strength parameters should be selected by the geotechnical engineer/geologist in consultation with the structural engineer. All parties must be aware of the implicit assumptions pertaining to the stability analysis procedure that is being used. The design strengths should be selected such that they do not result in uneconomical, ultra-conservative designs. However, in certain instances it may be more economical to assume conservative design shear strength parameters than to institute an expensive testing program. Selected strength parameters must be appropriate for the actual stress states and drainage conditions expected for the foundation materials. Laboratory results are dependent on the details of the testing procedures and the condition of the samples tested. The conversion from laboratory test data to in-situ strength parameters requires careful evaluation. A combination of experience and judgment is required to give the level of confidence needed for selecting the strength parameters. The shear strength of intact rock as well as rock with clean or filled discontinuities is dependent on many factors including confining pressures, loading history, and rate of loading. Specimen size is also a factor which must be considered when estimating shear strength based on laboratory testing. The number, orientation, and size of discontinuities and weaknesses may vary considerably, thus affecting load distribution and the final results. When selecting design shear strengths, the shape of stress-strain curves for individual tests should be considered. Where undisturbed and compacted samples do not show a significant drop in shear or deviator stress after the peak stress is reached, the design strength can be chosen as the peak shear stress. Where significant differences in stress-strain characteristics exist along a potential failure surface, stress-strain compatibility and the potential for progressive failure must be considered in selection of design strength parameters. With varied foundation conditions, it may not be possible to have all the foundation materials at their peak strengths at the same displacement (see Chapter 6). In those conditions, and for conditions that rely on passive resistance of a rock wedge or soil backfill, the strength values must be consistent with the displacements that will put the structure at the limit state assumed for the sliding stability analysis. A discussion of the sliding equilibrium method and its limitations can be found in EM 1110-1-2908.

2-10. Multiple-Wedge Sliding Analysis

a. Basic concepts. The multiple-wedge sliding analysis is a fairly simple assessment of the sliding factor of safety along various potential sliding planes, that can also account for the behavior expected from complex soil stratification and geometry. It is based on modern principles of structural and geotechnical mechanics that apply a safety factor to the material strength parameters in a manner, which places the forces acting on the structure and foundation wedges in sliding equilibrium. This method of analysis is illustrated in Appendix D, example D2. Derivation of the governing equilibrium equation for a typical wedge is shown in Appendix E. See EM 1110-1-2908 for additional information on this method. Following are the principles, assumptions and simplifications used in multiple-wedge sliding analysis.

- Sliding stability of most concrete structures can be adequately assessed by using a limit equilibrium approach.

- A sliding mode of failure will occur along a presumed failure surface when the applied shearing force exceeds the resisting shearing forces.

- The failure surface can be any combination of plane and curved surfaces, but for simplicity, all failure surfaces are assumed to be planes, which form the bases of wedges.

- Analyses are based on assumed-plane failure surfaces. The calculated safety factor will be realistic only if the assumed failure mechanism is possible.

- The factor of safety is defined, and minimum required factors of safety are given in Chapter 3.

- The lowest safety factor on a given failure surface can be determined by an iterative process. However, a single-step analysis using the required minimum factor of safety, can be used as a simple pass/fail test.

- A two-dimensional analysis is presented in this manual. These principles should be extended if unique three-dimensional geometric features and loads critically affect the sliding stability of a specific structure.

- Only force equilibrium is satisfied in this analysis, moment equilibrium is not ensured.

- The shearing force acting along the vertical interface between any two wedges is assumed to be negligible. Therefore, the failure surface at the bottom of each wedge is only loaded by the forces directly above it.

- A linear relationship is assumed between the resisting shearing force and the normal force acting on the failure surface beneath each wedge.

- The maximum shear strength that can be mobilized is adequately defined by the Mohr-Coulomb failure theory.

- Considerations regarding displacements are excluded from the limit equilibrium approach. The relative rigidity of different foundation materials and the concrete substructure may influence the results of the sliding-stability analysis. Such complex structure-foundation systems may require a more intensive sliding investigation than a limit equilibrium approach. The effects of strain compatibility along the assumed failure surface may be included by interpreting data from in situ tests, laboratory tests, and finite element analyses.

b. Analytical procedure. Following is a general procedure for analyzing multi-wedge systems.

- Assume a potential failure surface which is based on the stratification, location and orientation, frequency and distribution of discontinuities of the foundation material, and the configuration of the substructure. Discontinuities in the slip path beneath the structural wedge should be modeled by assuming an average slip plane along the base of the structural wedge. The structural wedge may include rock or soil that lies below the base of the concrete structure.

- Divide the assumed slide mass into a number of wedges. Since all portions of the structure must slide as a unit, there can be only a single structural wedge.

- The interface between the group of driving wedges (wedges with negative slip plane inclination angles) and the structural wedge is assumed to be a vertical plane located at the heel of the structural wedge and extending to the base of the structural wedge. The magnitudes of the driving forces depend on the actual values of the safety factor and the inclination angles (α) of the slip path. The inclination angles, corresponding to the maximum driving forces for each potential failure surface, can be determined by independently analyzing the group of driving wedges for a trial safety factor. In rock, the inclination may be predetermined by discontinuities in the foundation. The general equation applies to wedges with lateral earth forces that act horizontally.

- The interface between the group of resisting wedges (wedges with positive slip plane inclination angles) and the structural wedge is assumed to be a vertical plane located at the toe of the structural wedge and extending to the base of the structural wedge. The magnitudes of the resisting forces depend on the actual values of the safety factor and the inclination angles of the slip path. The inclination angles, corresponding to the minimum resisting forces for each potential failure mechanism, can be determined by independently analyzing the group of resisting wedges for a trial safety factor. When resisting force is used, special considerations may be required. Rock that may be subjected to high velocity water scouring should not be used unless adequately protected. Also, the compressive strength of the rock layers must be sufficient to develop the wedge resistance. In some cases, wedge resistance should not be assumed without resorting to special treatment such as installing rock anchors.

- Draw free body diagrams which show all the forces assumed to be acting on each wedge. The orientation of the failure surfaces for most wedges can be calculated directly by using the equations in paragraph 5-4.

- The analysis proceeds by assuming trial values of the safety factor and unknown inclinations of the slip path so the governing equilibrium conditions, failure criterion, and definition of safety factor are satisfied. An analytical or a graphical procedure may be used for this iterative solution.

- If it is only necessary to determine whether an adequate safety factor exists, this may be determined in a single step without the iterative process.

- For some load cases, the normal component of the resultant applied loads will lie outside the kern of the base area, and a portion of the structural wedge will not be in contact with the foundation material. The sliding analysis should be modified for these load cases to reflect the following secondary effects due to coupling of sliding and rotational behavior. The uplift pressure on the portion of the base, which is not in contact with the foundation material, should be a uniform value, which is equal to the hydrostatic pressure at the adjacent face, (except for instantaneous load cases such as due to seismic forces). The cohesive component of the sliding resistance should only include the portion of the base area, which is in contact with the foundation material.

c. Coordination. An adequate assessment of sliding stability must account for the basic structural behavior, the mechanism of transmitting compressive and shearing loads to the foundation, the reaction of the foundation to such loads, and the secondary effects of the foundation behavior on the structure. A fully coordinated team of geotechnical and structural engineers and geologists should ensure that the result of the sliding analyses is properly integrated into the overall design. Some of the critical aspects of the design process which require coordination are:

- Preliminary estimates of geotechnical data, subsurface conditions, and types of structures.

- Selection of loading conditions, loading effects, potential failure mechanisms, and other related features of the analytical models.

- Evaluation of the technical and economic feasibility of alternative structures.

- Refinement of the preliminary substructure configuration and proportions to consistently reflect the results of detailed geotechnical site explorations, laboratory testing, and numerical analyses.

- Modification of the structure configuration or features during construction due to unexpected variations in the foundation conditions.

2-11. Single-Wedge Sliding Analysis

Only the structural wedge is actively considered in the single-wedge sliding analysis. This is a simpler method, which will produce satisfactory results. The basic concepts are similar for both the single and multiple-wedge methods, but all driving and resisting wedges are replaced with earth and groundwater forces calculated directly, using the methods in Chapter 5. The single-wedge method is illustrated in Appendix D, example D1. This method produces reasonably conservative estimates of the earth forces used for the sliding analysis and for other stability analyses and for structural design.

2-12. Mandatory Requirements

There are no mandatory requirements in this chapter.

Chapter 3
Stability Requirements

3-1. General

The concepts used to develop the structural stability requirements contained in this manual are to establish safety factors or safety provisions for the three prescribed load condition categories of usual, unusual, and extreme such that the risk of a failure is kept to an acceptably low level and such that performance objectives are achieved. The use of three different design-load condition categories permits different safety factors or safety provisions to be assigned to the various load conditions depending on the probability of the load condition occurring during the life of the structure. The load conditions used in the stability analyses are described on a probabilistic basis, except the seismic loads and large flood loads falling into the extreme category may be either probabilistic or deterministic.

3-2. Load Condition Categories

The load conditions that a structure may encounter during its service life are grouped into the load condition categories of usual, unusual, and extreme. Associated with each category is a likelihood that the load condition will be exceeded in a given time period. The load conditions, expressed in probabilistic terms, are provided in Table 3-1. The structural performance and the risk of damage or failure depends not only on the likelihood of the loading condition, but also on the safety factors or the safety provisions used, the degree of conservatism used in selecting the foundation strength parameters and hydrological data, and the degree of conservatism inherent in the methods used for the analysis. No attempt has been made to define the likelihood of damage or failure in probabilistic terms. However, the use of these guidelines in conjunction with other Corps guidance will provide structures with adequate protection against stability failure.

Table 3-1 Load Condition Probabilities

Load Condition Categories	Annual Probability (p)	Return Period (t_r)
Usual	Greater than or equal to 0.10	Less than or equal to 10 years
Unusual	Less than 0.10 but greater than or equal to 0.0033	Greater than 10 years but less than or equal to 300 years
Extreme	Less than 0.0033	Greater than 300 years

- *Usual* loads refer to loads and load conditions, which are related to the primary function of a structure and can be expected to occur frequently during the service life of the structure. A usual event is a common occurrence and the structure is expected to perform in the linearly elastic range.

- *Unusual* loads refer to operating loads and load conditions that are of infrequent occurrence. Construction and maintenance loads, because risks can be controlled by specifying the sequence or duration of activities, and/or by monitoring performance, are also classified as unusual loads. Loads on temporary structures which are used to facilitate project construction, are also classified as unusual. For an unusual event some minor nonlinear behavior is acceptable, but any necessary repairs are expected to be minor.

- *Extreme* loads refer to events, which are highly improbable and can be regarded as emergency conditions. Such events may be associated with major accidents involving impacts or explosions and natural disasters due to earthquakes or flooding which have a frequency of occurrence that greatly exceeds the economic service life of the structure. Extreme loads may also result from the combination of unusual loading events. The structure is expected to accommodate extreme loads without experiencing a catastrophic failure, although structural damage which partially impairs the operational functions are expected, and major rehabilitation or replacement of the structure might be necessary.

Appendix B lists the loading conditions that must be evaluated to ensure the stability of specific structure types. The loading conditions have been taken from other USACE manuals and may have been modified to be consistent with other provisions of this manual. When a loading condition is defined in terms of a return period (for example, the Operational Basis Earthquake is defined as an earthquake with a return period of 144 years), the structural engineer can determine if the load condition is usual, unusual, or extreme by referring directly to Table 3-1. When a load condition is stated in non-probabilistic terms, (for example, pool elevation at the top of closed spillway gates, or water to the top of a flood wall), the return period must be determined to see if that particular load condition is usual, unusual, or extreme. In some cases, the load condition category is specifically designated based on established practice, irrespective of any return period (for example, construction is listed as an unusual loading). The engineer only needs to verify stability for those conditions listed in Appendix B. For example, for the unusual category, it is not necessary to verify stability for a 300 year flood or earthquake if these are not specifically listed in Appendix B. Definitions of common loadings for civil works projects are provided in Chapter 4, including: normal operating, infrequent flood, maximum design flood, probable maximum flood, operational basis earthquake, maximum design earthquake, and maximum credible earthquake.

3-3. Risk-based Analysis for USACE Flood Project Studies

USACE policy now requires the application of risk-based analysis in the formulation of flood-damage-reduction projects. The requirements are briefly discussed in the next paragraph to familiarize the structural engineer with the procedures used by hydrology/hydraulics (H&H) engineers use to develop the degree of protection provided by the project (i.e., dam height, floodwall height). The structural engineer needs to coordinate with the H&H engineers to obtain return periods for the required loading conditions to determine the load condition category from Table 3-1.

Risk-based analysis quantifies the uncertainty in discharge-frequency, elevation (stage)-discharge, and elevation-damage relationships and explicitly incorporates this information into economic and performance analyses of alternatives. The risk-based analysis is used to formulate the type and size of the optimal structural (or nonstructural) plan that will meet the study objectives. USACE policy requires that this plan be identified in every flood-reduction study it conducts. This plan, referred to as the National Economic Development Plan (NED), is the one that maximizes the net economic benefits of all the alternatives evaluated. It may or may not be the recommended plan, based on additional considerations. A residual risk analysis for the NED Plan is next performed to determine the consequences of exceeding project capacity. For any flood-protection project, it is possible that project capacity may be exceeded sometime during its service life. Therefore, the question becomes, “If that capacity is exceeded, what are the impacts, both in terms of economics and the threat to human life?” If the project-induced and/or residual risk is unacceptable, and a design to reduce the risk cannot be developed, other alternatives are further analyzed. Either a larger project, that will ensure sufficient time for evacuation, or a different type of project, with less residual risk, should be selected to reduce the threat to life and property. For a detailed discussion of the H&H requirements, see ER 1105-2-101 and EM 1110-2-1619.

When the type and size of the project have been selected, detailed design begins. The structural engineer, in coordination with the hydrology/hydraulic engineers, may use expected values (best estimates) of discharge-frequency and stage-discharge curves to estimate return periods for the various prescribed structure-dependent hydrostatic load conditions listed in Appendix B. For load conditions with prescribed water elevations, (for example, water to the top of closed spillway gates, or water to the top of a flood wall) the headwater elevation may be used in conjunction with the stage-discharge curve and discharge-frequency curves to estimate the annual probability and return period for the event representing the load condition. For some projects, such as high pools at power projects, other information such as project operating data will also be used in estimating the return period for a prescribed loading condition. The designer then refers to Table 3-1 to determine if each particular load condition is usual, unusual, or extreme.

3-4. Site Information

a. General. A proper stability analysis cannot be performed without knowing the potential planes of weakness beneath the structure, the strength of the materials along potential planes of weakness, uplift forces that occur on the structure or on planes of weakness, the strength of backfill materials, and all loads and load conditions to which the structure may be subjected. Knowledge of geologic formations beneath the structure is also important in defining seepage conditions and uplift pressures. Without adequate foundation explorations and testing, the safety factors provided to assess stability of the structure are meaningless. Preliminary stability analyses are useful to identify design parameters, which require special attention. In some rock foundations there may be many faults, shear zones, and discontinuities that make it impossible to do little more than predict average shear and cohesive strengths of the materials that make up the foundation. Use of lower bound values for foundation shear strength or upper bound values for loads is only acceptable when it can be demonstrated that the added costs to improve the accuracy of the strength and loading data will not lead to significant savings for the structure or foundation. Lower factors of safety are permitted by this manual in cases where there is an abundance of information on the various foundation and structure properties used to establish the strength parameters for the stability analysis. Conversely, higher factors of safety are required when there is only limited information on either foundation or structure properties. Three categories of site information, *well defined, ordinary*, and *limited,* were used in establishing safety requirements.

b. Well-defined site information. This category is restricted to use for existing projects. To qualify as well defined, site information must satisfy the following requirements:

- Available records of construction, operation, and maintenance indicate the structure has met all performance objectives for the load conditions experienced.
- Foundation strengths can be established with a high level of confidence.
- The governing load conditions can be established with a high level of confidence.
- Uplift pressures for design load conditions are known, or can be extrapolated for design load conditions based on measured uplift pressure data.

c. Ordinary site information. This category applies to most new project designs. To qualify as ordinary, site information must satisfy the following requirements:

- Foundation strengths have been established with current USACE explorations and testing procedures.
- Foundation strengths can be established with a high level of confidence.
- The governing load conditions can be established with a high level of confidence.

d. Limited site information. This category applies to those new or existing structures designated as *normal* (*critical* structures can not be designed or evaluated based on *limited* site information), where either of the following are true:

- Foundation strengths are based on limited or inadequate explorations and testing information, or
- Governing load conditions cannot be established with a high level of confidence because of insufficient historical data on stream flow, flood potential, etc.

3-5. Critical Structures

Civil works structures, for the purpose of establishing safety factors or safety provisions for use in stability analyses, are to be designated as either critical or normal. Structures designated as critical are those structures on high hazard projects whose failure will result in loss of life. Loss of life can result directly, due to flooding or indirectly from secondary effects. Loss of life potential should consider the population at risk, the downstream flood wave depth and velocity, and the probability of fatality of individuals within the affected population. Information is provided in Appendix H to help design engineers determine if the structure should be designated critical or normal.

3-6. Existing Structures

The safety factors provided in this manual are based on the assumption that for critical and normal structures, the strength of the materials in the foundation and structure has been conservatively established through explorations and testing. This may not be the case for older existing structures, or, if adequate explorations and testing were performed, the records may not be available. When the stability of an existing structure is in question, a phased, systematic approach to evaluating stability should be performed before any remedial actions are undertaken to improve stability. This systematic evaluation process is described in Chapter 7. The load conditions used to evaluate an existing structure should be carefully checked to make sure that what was considered as a usual load condition for the original design is not, once the probabilities of the load conditions are examined, really an unusual or extreme load condition. Evaluation of existing structures should utilize analytical methods which accurately describe the behavior without introducing excess conservatism. When available, actual uplift pressures can be used as a basis for evaluating the stability of existing structures.

3-7. Factors of Safety for Sliding

Analysis of sliding stability is discussed in detail in Chapter 2 and Chapter 5. A factor of safety is required in sliding analyses to provide a suitable margin of safety between the loads that can cause instability and the strength of the materials along potential failure planes that can be mobilized to prevent instability. The factor of safety for sliding is defined by equation 3-1. The required factors of safety for sliding stability for *critical* structures and for *normal* structures are presented in Tables 3-2 and 3-3, respectively.

$$FS_s = \frac{N \tan\phi + cL}{T} \tag{3-1}$$

where

N = force acting normal to the sliding failure plane under the structural wedge.
ϕ = angle of internal friction of the foundation material under the structural wedge.
c = cohesive strength of the foundation material under the structural wedge.
L = length of the structural wedge in contact with the foundation.
T = shear force acting parallel to the base of the structural wedge.

Table 3-2 Required Factors of Safety for Sliding - Critical Structures

	Load Condition Categories		
Site Information Category	**Usual**	**Unusual**	**Extreme**
Well Defined	1.7	1.3	1.1
Ordinary	2.0	1.5*	1.1*
Limited**	-	-	-

*For preliminary seismic analysis without detailed site-specific ground motion, use FS=1.7 for unusual and FS=1.3 for extreme. See further explanation in section 3.11 b.
**Limited site information is not permitted for critical structures

Table 3-3 Required Factors of Safety for Sliding - Normal Structures

	Load Condition Categories		
Site Information Category	**Usual**	**Unusual**	**Extreme**
Well Defined	1.4	1.2	1.1
Ordinary	1.5	1.3	1.1
Limited	3.0	2.6	2.2

3-8. Factors of Safety for Flotation

A factor of safety is required for flotation to provide a suitable margin of safety between the loads that can cause instability and the weights of materials that resist flotation. The flotation factor of safety is defined by equation 3-2. The required factors of safety for *flotation* are presented in Table 3-4. These flotation safety factors apply to both *normal* and *critical* structures and for all site information categories.

$$FS_f = \frac{W_S + W_C + S}{U - W_G} \tag{3-2}$$

where

W_S = weight of the structure, including weights of the fixed equipment and soil above the top surface of the structure. The moist or saturated unit weight should be used for soil above the groundwater table and the submerged unit weight should be used for soil below the groundwater table.
W_C = weight of the water contained within the structure
S = surcharge loads
U = uplift forces acting on the base of the structure
W_G = weight of water above top surface of the structure.

Table 3-4 Required Factors of Safety for Flotation – All Structures

	Load Condition Categories		
Site Information Category	**Usual**	**Unusual**	**Extreme**
All Categories	1.3	1.2	1.1

3-9. Limits on Resultant Location

The factor of safety approach established for sliding and flotation is not appropriate for use in the evaluation of rotational modes of failure. Rotational behavior is evaluated by determining the location of the resultant of all applied forces with respect to the potential failure plane. This location can be determined through static analysis. Limits on the location of the resultant are provided in Table 3-5. The entire base must be in compression for the usual load condition, to maintain full contact between the structure and the foundation, so there is no chance for higher uplift pressures to develop in a crack. This helps ensure linear behavior for common loading conditions. For the unusual load case, higher uplift pressures may develop in a relatively short crack, but this would cause only minor nonlinear behavior. For extreme load conditions on typical civil works projects, a shear or bearing failure will occur before overturning could occur. Therefore, the resultant is permitted to be anywhere within the base, and safety is ensured by the safety factor requirements for sliding and by the limits on allowable bearing stresses.

Table 3-5 Requirements for Location of the Resultant – All Structures

	Load Condition Categories		
Site Information Category	**Usual**	**Unusual**	**Extreme**
All Categories	100% of Base in Compression	75% of Base in Compression	Resultant Within Base

3-10. Allowable Bearing Capacity

Allowable concrete compressive stresses and/or allowable bearing capacity values established by materials engineers and geotechnical engineers are used as the basis for evaluating bearing modes of failure. The allowable bearing capacity value is defined as the maximum pressure that can be permitted on soil or rock giving consideration to all pertinent factors with adequate safety against rupture of the soil or rock mass, or movement of the foundation of such magnitude that the structure is impaired. Bearing failure is related to the relative compressibility of the foundation materials, the loading conditions, the geometry of the structure base, and the strength of the foundation and concrete at the structure-foundation interface. Bearing capacity may be related to the shear capacity of the foundation materials or to the deformability of the foundation. Information on foundation bearing analysis can be found in EM 1110-1-1905 for soils, and EM 1110-1-2908 for rock. Safety against bearing failure is generally expressed in terms of an allowable compressive stress for concrete and an allowable bearing capacity for foundation materials. These allowables include an adjustment, which represents a factor of safety. The allowable compressive stress and allowable bearing capacity values are established by testing performed by materials engineers and geotechnical engineers. Discussion on exploration and testing can be found in Chapter 2. The allowable compressive stress and bearing capacity values established for usual load conditions can be increased for the unusual and extreme load conditions. A 15% increase is permitted for unusual load conditions and a 50% increase is permitted for extreme load conditions.

3-11. Seismic Stability

a. General. Traditionally, the seismic coefficient method has been used to evaluate the stability of structures subjected to earthquake ground motions, but this method fails to take into account the true dynamic characteristics of the structure. There have been cases where structures similar to those used on civil works projects have failed during earthquakes because of a sliding or bearing failure. These failures for the most part are attributable to liquefaction and soil strength degradation in the foundation or backfill materials. Seismic stability analyses should be a performed in phases in accordance with requirements of ER 1110-2-1806. Seismic loads to be used in the first phase analysis are provided in Chapter 5 of this manual. Structures which meet sliding stability factor of safety requirements when evaluated by this procedure are considered to be safe and no additional seismic stability analyses are required. Structures that fail to meet factor of safety requirements when evaluated using this procedure should not be considered unsafe or in need of a stability retrofit. The failure to meet these requirements should only suggest the need for other seismic coefficient and dynamic analyses to fairly assess the demands placed on the structure and foundation during a major earthquake. From these advanced analyses engineers can determine if the displacements and stresses experienced by the structure and foundation will place the structure at risk of a stability failure. In many instances, it is acceptable for sliding and rocking to occur at the base of the structure during extreme earthquake load conditions. Stability in such cases is evaluated using dynamic analysis methods, and performance is ensured by limiting permanent displacements to acceptable levels.

b. Modified Factor of Safety. The factors of safety given in Tables 3-2 include FS=1.5 for unusual and FS = 1.1 for extreme load conditions, for ordinary site information. The ordinary site information and related factor of safety must be used in the seismic coefficient method. These factors of safety are based on use of extreme loads with very low probabilities of being exceeded. When factors of safety for seismic loadings are being calculated using the coefficient method, the MCE loads are usually not based on detailed site-specific seismic data. Since the loads would be based on less precise data, there would be greater probability that the predicted extreme loads could be exceeded, therefore, it is appropriate to use higher factor of safety for such analyses. For such analyses, use a factor of safety of 1.7 for unusual and 1.3 for extreme, as stated in the notes following the above table.

3-12. Mandatory Requirements

For a general discussion on mandatory requirements, see Paragraph 1-5. As stated in that paragraph, certain requirements within this manual are mandatory. The following are mandatory for Chapter 3.

a. Load condition categories. Unless the loading condition category (usual, unusual, extreme) is specifically designated in Appendix B, the return period range limitations specified in Table 3-1 shall be used to establish the correct loading condition designation. When the return period for a particular loading condition can not be established with sufficient accuracy to determine if the loading condition is usual or unusual (or unusual or extreme), the loading condition with the more stringent safety requirements shall be used.

b. Critical structures. In accordance with section 3-5, structures on high hazard projects shall be considered *critical* where failure will result in loss of life; all other structures will be classified as *normal*. In making the determination of critical or normal, the engineer must follow the guidelines in Appendix G.

c. Site information. Structures shall be assigned to one of three site information categories: *well-defined*, *ordinary*, or *limited*. Site information category selection shall be in accordance with the provisions of Paragraph 3-4.

d. Sliding stability. Sliding stability factors of safety shall be equal to, or greater than, the values specified in Tables 3-2 and 3-3. The sliding stability factor of safety shall be determined using Equation 3-1.

e. Flotation stability. Flotation factors of safety shall be equal to, or greater than, the values specified in Table 3-4. The flotation stability factor of safety shall be determined using Equation 3-2.

f. Resultant location. The location of the resultant of all forces acting on the base of the structure shall be within the limits specified in Table 3-5.

g. Bearing pressures. Bearing pressures for *usual* load conditions shall be within allowable limits established by the geologist/geotechnical engineer. Increases in allowable bearing pressures shall not exceed 15% for *unusual* and 50% for *extreme* load conditions, in accordance with the guidance in section 3-10.

h. Loading conditions. As a minimum, the loading conditions provided in Appendix B shall be satisfied in the stability analysis.

i. Loads. Loads shall comply with the mandatory requirements of Chapters 4 and 5.

Chapter 4
Loads and Loading Conditions

4-1. General

Previously, stability criteria was provided in separate manuals for each type of structure. Those manuals listed all of the load cases (loading conditions), which had to be investigated as part of the stability analysis. Those loading conditions are now summarized in tables provided in Appendix B of this manual. The tables list the loading condition and give a classification as usual, unusual, or extreme, as defined in Table 3-1. Following each table are brief descriptions of the loading conditions. The loading conditions have been revised in some cases for general consistency with the provisions of this manual, especially to comply with current practice for flood and seismic loadings. This chapter defines most of the types of loads that are combined to form each loading condition. However, soil loads are defined in Chapter 2 for multiple wedge sliding analyses and in Chapter 5 for single wedge sliding analyses.

4-2. Construction

Based on past practice, construction loading conditions shall be classified as *unusual*, regardless of duration.

4-3. Water Loading Conditions

a. General. All water loading conditions should be based on hydrologic information, which gives median water elevations in terms of return periods. A typical flood hazard curve is illustrated in Figure 4-1. Curves for both headwater and coincident tailwater will be necessary to determine the water loads for dams and navigation locks. Hydraulic engineers commonly use the 90-percent confidence level hazard curve when determining flood protection requirements. However, for stability analysis, structural engineers require median flood hazard curves, which can also be provided by the hydraulic engineers. Based on the information presented in Figure 4-1 a flood pool elevation equal to 21 meters (68.9 feet) would be used to determine the maximum *unusual* loading.

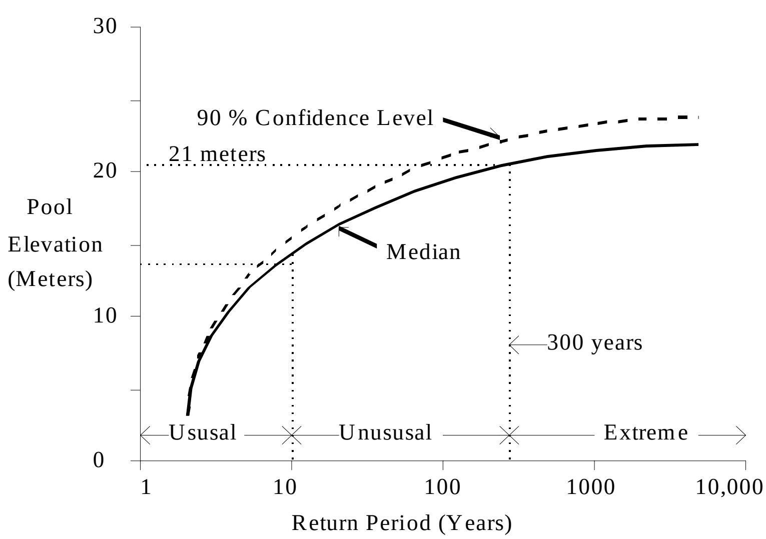


Figure 4-1 Flood Hazard Curve

b. Coincident pool. Coincident pool represents the water elevation that should be used for combination with seismic events. It is the elevation that the water is expected to be at or below for half of the time during each year.

c. Normal operation. In the past, a normal operation loading condition has been used to describe loadings with various probabilities of occurring, including rare events with long return periods. To be consistent with Table 3-1, normal operating conditions are now defined as maximum loading conditions with a return period of no more than 10 years (annual probability of 10%). For certain floodwalls, this means that there might be no water loads on the structure for normal operation. For hydropower dams, the pool will be fairly high for normal operation, while for some flood-control dams, the pool will be low for normal operation. For navigation projects, the maximum loading for normal operation might correspond to the usual navigation pool, combined with the lowest tailwater expected with a 10-year return period. Water loads defined by the normal operation loading condition are sometimes combined with other types of events (such as barge impacts).

d. Infrequent flood. The infrequent flood (IF) represents flood pool or water surface elevations associated with events with a return period of no greater than 300 years (annual probability of 0.33%), making the IF an unusual loading per Table 3-1. This loading condition replaces loadings such as water *to top of spillway gates* and *water to spillway crest* previously used for the design and evaluation of gated and ungated spillways. It also replaces the *design flood* (top of wall less freeboard) used for the design and evaluation of floodwalls. In limited cases, historical hydrologic data may be inadequate to determine the 300-year water elevations with reasonable certainty. In such cases, traditional loading conditions such as *water to top of spillway gates*, *water to spillway crest*, and *design flood* shall be considered unusual events and evaluated in addition to the IF event.

e. Probable maximum flood. The probable maximum flood (PMF) is one that has flood characteristics of peak discharge, volume, and hydrograph shape that are considered to be the most severe reasonably possible at a particular location, based on relatively comprehensive hydro-meteorological analyses of critical runoff-producing precipitation, snow melt, and hydrologic factors favorable for maximum flood runoff. The PMF load condition represents the most severe hydraulic condition, but because of possible overtopping and tailwater effects, it may not represent the most severe structural loading condition, which is represented by the maximum design flood described below. Therefore, the PMF condition will not necessarily be examined for structural stability.

f. Maximum design flood. The maximum design flood (MDF) is the designation used to represent the maximum structural loading condition (as judged by the minimum factor of safety) and must be determined for each structure or even for each structural element. MDF may be any event up to PMF. For floodwalls, MDF is usually when the water level is at or slightly above the top of the wall. Overtopping from higher water levels would result in rising water levels on the protected side, thus reducing net lateral forces. The same situation may be true for dams, but often significant overtopping can occur without significant increases in tailwater levels. The design engineer must consult with the hydraulics engineer to explore the possible combinations of headwater and tailwater and their effects on the structure. Some elements of dam outlet works (such as chute walls or stilling basins) are loaded differently from the main dam monoliths. For such elements, different flow conditions will produce maximum structural loading. When it is not obvious which loading will produce the lowest factor of safety, multiple loadings should each be investigated as a possible MDF. Since sliding is the most likely mode of failure for most gravity structures, MDF can usually be judged by determining maximum net shear forces. However, due to variable uplift conditions, a loading with smaller shears could result in the lowest factor of safety. Once the MDF is determined, it should be classified as usual, unusual, or extreme per Table 3-1, based on its return period.

4-4. Uplift Loads

Uplift loads have significant impact on stability. Sliding stability, resultant location, and flotation are all aspects of a stability analysis where safety can be improved by reducing uplift pressures. Since uplift pressures are directly related to flow paths beneath the structure, uplift pressure distribution may be determined from a seepage analysis. Such an analysis must consider the types of foundation and backfill materials, their possible range of horizontal and vertical permeabilities, and the effectiveness of cutoffs and drains. Techniques for seepage analysis are discussed in EM 1110-2-1901, EM 1110-2-2502, Casagrande (1937), Cedergren (1967), Harr (1962), and EPRI (1992). Seepage analysis techniques to determine uplift pressures on structures include flow nets, finite element methods, the line-of-creep method, and the method of fragments. Uplift pressures resulting from flow through fractures and jointed rock, however, are poorly understood and can only be accurately known by measurements taken at the point of interest. Joint asperities, changes in joint aperture, and the degree to which joints interconnect with tailwater influence uplift pressures and pressure distribution. Uplift pressures are site-specific and may vary at a given site due to changes in geology. Uplift pressures can be reduced through foundation drainage, or by various cutoff measures such as grout curtains, cutoff walls, and impervious blankets. Uplift pressures should be based on relatively long-term water elevations. Short duration fluctuations, such as from waves or from vibrations due to high velocity flows, may be safely assumed to have no effect on uplift pressures. Uplift pressures to be used for stability analysis of new structures are covered in Appendix C. The conservative uplift pressures used for the design of new structures may be significantly higher than those the actual structure may experience during its lifetime. For this reason, the use of actual uplift pressures for the evaluation of existing structures is permitted under the provisions discussed in Chapter 7. However, the engineer should be aware that in some instances the actual uplift may not be reflected by uplift cell readings. Since uplift measurement devices only capture a snapshot of a given part of the foundation, they should be used with caution, based on an overall evaluation of the foundation.

4-5. Maintenance Conditions

The return periods for a maintenance condition loading may be greater or less than 10 years, but based on past experience maintenance has been designated as an *unusual* load condition. The classification as an *unusual* loading is based on the premise that maintenance loadings take place under controlled conditions and that the structure performance can be closely monitored during maintenance.

4-6. Surge and Wave Loads

a. General. Surge and wave loads are critical in analyzing the stability of coastal protection structures but usually have little effect on the stability of inland structures. Wave and water level predictions for the analysis of structures should be based on the criteria presented in the Shore Protection Manual 1984, EM 1110-2-1612, and EM 1110-2-1614. Design forces acting on the structure should be determined for the water levels and waves predicted for the most severe fetch and the effects of shoaling, refraction, and diffraction. The methods recommended for calculation of wave forces are for vertical surfaces. Wave forces on other types of surfaces (sloping, stepped, curved, etc.) are not sufficiently understood to recommend general analytical design criteria. In any event, the structural engineer should consult with a coastal engineer in establishing wave forces for the design of critical structures.

b. Wave heights. Wave heights for design are obtained from the statistical distribution of all waves in a wave train and are defined as follows:

H_S = average of the highest one-third of all waves

$H_1 = 1.67\ H_S$ = average of highest 1 percent of all waves

H_b = height of wave which breaks in water depth d_b

c. Non-breaking waves. When the water depth is greater than approximately 1.5 times the wave height , waves do not break. The H_1 wave shall be used for the non-breaking condition. Design pressures for non-breaking waves shall be computed using the Miche-Rudgren method. Whenever the maximum stillwater level results in a non-breaking condition, lower stillwater levels should be investigated for the possibility that shallow water may produce breaking wave forces, which are larger than the non-breaking forces.

d. Breaking waves. Waves break when the steepness of the wave and the bottom slope at the front of the structure have certain relationships to each other. It is commonly assumed that a wave will break if the water depth is not greater than 1.3 times the wave height. Study of the breaking process indicates that this assumption is not always valid. The height of the breaking wave and its breaking point are difficult to determine, but breaker height can equal the water depth at the structure, depending on bottom slope and wave period. Detailed determination of breaker heights and distances for a sloping approach grade in front of the structure are given in the Shore Protection Manual 1984. Design breaking wave pressure should be determined by the Minikin method presented in EM 1110-2-1614. Breaking-wave impact pressures occur at the instant the vertical face of the wave hits the structure and only when a plunging wave entraps a cushion of air against the structure. Because of this dependence on curve geometry, high impact pressures are infrequent against prototype structures; however, they must be recognized as possible and must be considered in design. Also, since the impact pressures caused by breaking waves are of very short duration, their importance in design against sliding and rotational instability may be questionable relative to longer lasting, smaller dynamic forces.

e. Broken waves. Broken waves are those that break before reaching the structure, but near enough to have retained some of the forward momentum of breaking. The design breaker height in this case (H_b) is the highest wave that will be broken in the breaker zone. Design wave forces for this height should be determined by the method presented in Chapter 7 of the Shore Protection Manual (1984).

4-7. Earthquake Loading Conditions

a. Seismic Load Conditions. Earthquake loads are used to represent the inertial effects attributable to the structure mass, the surrounding soil (dynamic earth pressures), and the surrounding water (hydrodynamic pressures). Design earthquakes shall comply with requirements of ER 1110-2-1806, based on the following seismic events.

- *Operational basis earthquake (OBE).* The OBE is considered to be an earthquake that has a 50 percent chance of being exceeded in 100 years (or a 144-year return period).

- *Maximum design earthquake (MDE).* The MDE is the maximum level of ground motion for which a structure is designed or evaluated. For *critical* structures the MDE is the same as the maximum credible earthquake (MCE). Generally, the probabilistically determined MDE for other structures is an earthquake that has a 10 percent chance of being exceeded in a 100-year period (or a 950-year return period).

- *Maximum Credible Earthquake.* The MCE is defined as the greatest earthquake that can reasonably be expected to be generated on a specific source, on the basis of seismological and geological evidence. The MCE is based on a deterministic site hazard analysis.

Earthquake-generated inertial forces associated with the OBE are unusual loads. Those associated with the MDE are extreme loads. Earthquake loads are to be combined with other loads that are expected during routine operations, and should not be combined with other infrequent events such as flood loads. Seismic loads should be combined with *coincident pool*, which is defined as the elevation that the water is expected to be at or below for half of the time during each year.

b. Analytical Methods. Several analytical methods are available to evaluate the dynamic response of structures during earthquakes: seismic coefficient, response spectrum, and time-history. These methods are discussed in reference ER 1110-2-1806. The current state-of-the-art method uses linear-elastic and nonlinear finite element time history analysis procedures, which account for the dynamic interaction between the structure, foundation, soil, and water. The seismic coefficient method, although it fails to account for the true dynamic characteristics of the structure-water-soil system, is accepted as a semiempirical method for determining if seismic forces control the design or evaluation, and to decide if dynamic analyses should be undertaken. The information in the following paragraphs describes the differences between the seismic coefficient method and dynamic analysis methods. Figure 4-2 illustrates the differences in the inertial and hydrodynamic earthquake loads obtained by the two different methods. The seismic coefficient used for the preliminary seismic stability evaluation of concrete hydraulic structures should be equal to 2/3 the effective peak ground acceleration (EPGA) expressed as a decimal fraction of the acceleration of gravity. The EPGA can be obtained by dividing the 0.30 second spectral acceleration, for the return period representing the design earthquake, by a factor of 2.5 The 0.30 second spectral acceleration is obtained from the spectral acceleration maps in Appendix D of ER 1110-2-1806.

c. Inertia force due to structure mass. In the seismic coefficient approach, the inertial force is computed as the product of the mass of the structural wedge (including the soil above the heel or toe and any water contained within the structure) and the seismic acceleration. This may also be expressed as the weight of the structural wedge times the seismic coefficient, expressed as a fraction of gravity.

$$F_h = m\,a = k_h\,W \qquad (4\text{-}1)$$

where: F_h = horizontal component of the inertial force (a similar equation can be used for vertical component)
m = mass of structural wedge
a = seismic acceleration
W = gross weight of structural wedge (including soil above the heel and toe, and water contained within the structure)
k_h = seismic coefficient = a / g
g = acceleration of gravity

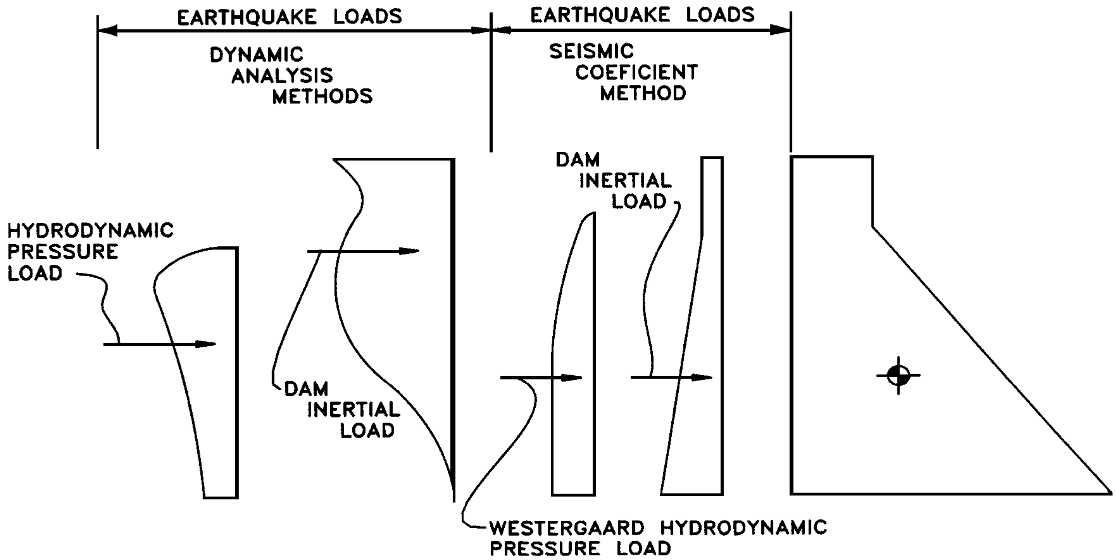


Figure 4-2 Idealized earthquake loads

The horizontal component of the inertial force is assumed to act at the center of mass of the structure, based on the assumption that the structure is a rigid body. In actuality, almost all structures have some flexibility, and the use of the rigid body concept often under estimates the magnitude of the inertial force. The location of the horizontal inertial force is also related to the flexibility of the structure, and usually acts at a location higher than the center of mass. However, because of the cyclic nature of earthquake loads, there is little probability of a rotational-stability related failure.

d. Inertial effects of soil. Backfill material adjacent to a structure will induce inertial forces on the structure during an earthquake. See Chapter 5 and Appendix G for information on soil loads due to earthquakes.

e. Effects of water. Water that is above the ground surface and adjacent to, or surrounding a structure will increase the inertial forces acting on the structure during an earthquake. The displaced structure moves through the surrounding water thereby causing hydrodynamic forces to act on the structure. The water inside and surrounding the structure alters the dynamic characteristics of the structural system, increasing the periods of the fundamental modes of vibration and modifying the mode shapes. In seismic coefficient methods, the hydrodynamic effects are approximated by using the Westergaard method (equation 4-2) (Westergaard 1933). The hydrodynamic force can either increase or decrease the water force, depending on direction of seismic acceleration. Figure 4-3 illustrates hydrodynamic pressures based on the Westergaard method.

$$P_E = (7/12)\ kh\ \gamma_w\ h^2 \qquad (4\text{-}2)$$

where: P_E = hydrodynamic force per unit length
k_h = horizontal seismic coefficient
γ_w = unit weight of water
h = water depth.

The hydrodynynamic force is added algebraically to the static water pressure force to get the total water force on the structure. The pressure distribution is parabolic and the line of action for the force P_E is 0.4 h above the ground surface. The Westergaard method assumes the structure is rigid and the water is incompressible. Since most structures are flexible, this method can lead to significant error. For free-standing intake towers, the hydrodynamic effects are approximated by adding mass to the structure to represent the influence of the water inside and surrounding the tower. Engineers using the seismic coefficient approach for stability analyses should be aware of the limitations and the simplifying assumptions made with respect to hydrodynamic pressures and their distribution on the structure.

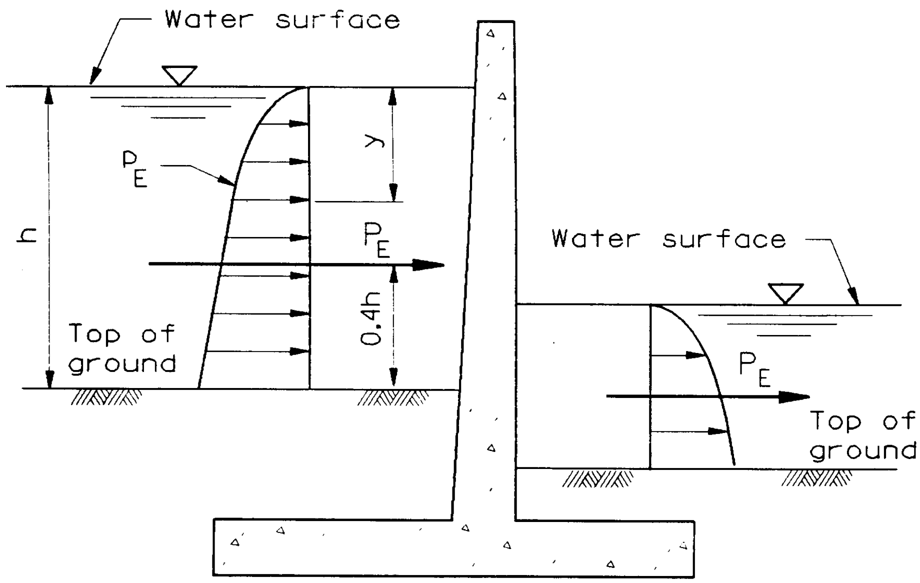


Figure 4-3 Hydrodynamic Forces for Freestanding Water

4-8. Other Loads

a. Impact. Impact loads for locks and dams on navigation systems are due to the structures being struck by barges. These loads can be quite large and for some structures, such as lock guide walls, control the stability analyses. Where impact loads must be considered, refer to EM 1110-2-2602.

b. Ice. Loads due to ice are usually not critical factors in the stability analysis for hydraulic structures. They are more important in the design of gates and other appurtenances. Ice damage to gates is quite common, but there is no known case of a dam failure due to ice. Where ice loads must be considered, refer to EM 1110-2-1612.

c. Debris. Debris loads, like ice loads, are usually of no consequence in stability analyses. However, they may be critical for the design of gates and floodwalls.

d. Hawser pull. Hawser pulls from barges are significant in the stability analysis for lock guide walls, mooring facilities, and floodwalls. Where hawser pulls must be considered, refer to EM 1110-2-2602.

e. Wind. Wind loads are usually small in comparison to other forces, which act on civil works structures. Therefore, wind loads should usually be ignored. For structures such as coastal flood walls where wind might cause instability, or for structures under construction, wind pressures should be based on the requirements of ASCE 7

f. Silt. Silt accumulation can occur upstream of dams. Not all dams will be susceptible to silt accumulation and the structural engineer should consult with hydraulic engineers to determine if silt buildup is possible, and to what extent it may accumulate over time. Silt loads should be included in the loading conditions indicated in Appendix B. Horizontal silt pressure is assumed to be equivalent to that of a fluid weighing 1362 kg/m^3 (85 pcf). Vertical silt pressure is determined as if silt were a soil having a wet density of 1922 kg/m^3 (120 pcf). These values include the effects of water within the silt.

4-9. Mandatory Requirements.

For a general discussion on mandatory requirements, see Paragraph 1-5. As stated in that paragraph, certain requirements within this manual are mandatory. The following are mandatory for Chapter 4.

a. Load Conditions. Stability shall be satisfied for all load conditions listed in Appendix B.

b. Maintenance. Maintenance load conditions shall be classified as *unusual*.

c. Water loading conditions. Water loadings shall be based on hydrologic analyses giving median water elevations in terms of return periods. Water elevations for various load conditions shall be as follows:

- *Coincident pool* shall be the elevation that the water is expected to be at or below for half of the time during each year. This water loading shall be used in combination with seismic loads.
- *Normal operation* loading shall represent maximum loads with a 10-year return period.
- *Infrequent flood* shall represent maximum loads with a 300-year return period.
- *Maximum design flood* shall be the maximum structural loading up to PMF.

d. Uplift loads. Uplift loads shall be calculated per the requirements of Appendix C, as follows:

- Uplift pressures shall be calculated based on an approximate seepage analysis and shall be applied over the full area of the base of the structure, or the failure plane under investigation.
- When a loss of contact is calculated to occur at the heel of the structure, full uplift pressure due to headwater shall be assumed to exist in this area. This provision does not apply to earthquake loading conditions.
- The maximum assumed effectiveness of drainage systems, cutoff wall systems, and combined drain and cutoff wall systems shall be 50%.
- Where overflow results in significant velocities and causes hydraulic jump and retrogression, tailwater pressures used in uplift calculations shall be reduced as described in Appendix C.

e. Earthquake loads.

- Earthquake loads shall be based on design earthquakes specified in ER 1110-2-1806.
- Structural inertia loads shall be calculated using the seismic coefficient method.
- Hydrodynamic loads shall be calculated using Westergaard's formula.
- Soil loads for seismic events shall be calculated per the requirements of Chapter 5.

Chapter 5
Soil Forces and Single-Wedge Sliding Analysis

5-1. General

Chapter 4 described various loading conditions and specific loads, except for soil loads. Chapter 5 describes soil loads and explains how to use various loads in a single-wedge stability analysis. The methods presented in this chapter are intended to produce reasonably conservative estimates of soil forces acting on a structure. This manual only addresses normal soil conditions, other conditions such as swelling soils require special studies. The definitions of terms that will be used throughout this chapter are as follows:

- *Single wedge.* The single wedge is the wedge to which forces are applied, i.e., the structure itself, which is referred to as the structural wedge.

- *Applied driving forces.* Driving forces are defined as those lateral forces whose primary influence is to decrease structural stability. The side of the structure upon which these forces are applied will be called the driving side. Uplift and downdrag are also treated as applied forces.

- *Applied resisting force*s. Resisting forces are defined as those lateral forces whose primary influence is to increase structural stability. The side of the structure upon which these forces are applied will be called the resisting side. The resisting side is on the opposite side of the structural wedge from the driving side. The difference between the driving and resisting forces is transferred to the foundation by the structural wedge.

- *Reactions.* The shear and the normal force between the foundation and the base of the single wedge are reactions, which are necessary to place the structure in static equilibrium, they are not included in the applied forces.

5-2. Single-Wedge Stability Analyses

a. Basic requirements. For the single-wedge analysis, the engineer must calculate the driving and resisting soil forces, lateral water forces, and uplift and apply them to the structural wedge. The vertical drag force discussed in Appendix F may also be included, if the requirements stated in the appendix are satisfied. Using these forces and the weight of the structural wedge, the engineer can determine the magnitude, location, and slope of the resultant acting at the base of the structural wedge. The resultant will be used to evaluate sliding stability, and the location of the resultant will be used to determine the potential for partial loss of contact between the structure and the foundation materials. The resultant and its location will also be used to evaluate the bearing capacity of the foundation materials. The forces applied to the structural wedge will also be used for design of the structural elements (e.g., shears, moments and axial loads in the base and stem of a retaining wall).

b. Soil forces. Lateral soil forces acting on the single wedge should be calculated using the minimum required factor of safety against sliding to obtain the developed soil strength parameters ϕ_d and c_d. The use of developed parameters results in a force larger than the active soil force on the driving side, and a force smaller than the passive soil force on the resisting side These developed parameters shall be used in Equations 5-3 through 5-15 to calculate the lateral soil forces acting on the driving side, and used in Equations 5-16 through 5-22 to calculate the lateral soil forces acting on the resisting side. Soil forces due to seismic events are discussed in Section 5-5. If there is a significant difference between the calculated and minimum required safety factors, it may be appropriate to re-evaluate the developed soil strength parameters used to determine the soil forces. The values for the developed soil strength parameters shall be determined as:

$$\phi_d = \tan^{-1}\left(\frac{\tan\phi}{FS_s}\right) \quad and \quad c_d = \frac{c}{FS_s} \qquad (5\text{-}1,\ 5\text{-}2)$$

where FS_s = required factor of safety against sliding

ϕ = nominal angle of internal friction of the soil

c = nominal cohesive strength of the soil

c. *Sliding.* The resultant can be resolved into components parallel and normal to the base plane of the structural wedge. The sliding factor of safety is calculated as follows:

$$FS_s = \frac{N \tan \phi + c\,L}{\mathrm{T}} \tag{5-3}$$

where: N = the component of the resultant normal to the base

T = the component of the resultant parallel to the base

L = length of base in compression

If the safety factor is equal to or greater than the required safety factor, the sliding stability criterion is satisfied. Note that this calculated safety factor might not be equal to the minimum factor used to determine developed soil strength parameters. If there is a significant difference between the calculated and minimum required safety factors, it may be appropriate to re-evaluate the developed soil strength parameters used to determine the soil forces. The resultant location shall be used to determine the length of the base that is in compression, and cohesion shall not be effective on that part of the base that is not in compression. If there is any loss of contact, uplift forces should be re-evaluated. However, for seismic events, cyclic loading periods are so short that the time is not sufficient for uplift pressures to change, therefore, uplift should not be increased due to loss of contact for seismic loads.

5-3. Soil Pressures and Forces

a. Active soil pressures.

(1) Cohesionless backfill. Cohesionless materials such as clean sand are the recommended backfill for most structures. Large-scale tests (Terzaghi 1934; Tschebatarioff 1949; Matsuo, Kenmochi, and Yagi 1978) with cohesionless backfills have shown that lateral pressures are highly dependent on the magnitude and direction of wall movement. The minimum lateral pressure condition, or active soil pressure, develops when a structure rotates about its base and away from the backfill an amount on the order of 0.001 to 0.005 radians. As the structure moves, horizontal stresses in the soil are reduced, and vertical stresses due to backfill weight are resisted by increasing shear stresses until shear failure is imminent (Figures 5-1 and 5-2).

(2) Cohesive backfill. For situations where cohesive backfill is unavoidable, solutions are included herein for soil pressures involving both frictional and cohesive soil strength parameters (ϕ and c). Where cohesive backfill is used, two analyses (short-term and long-term) are usually required in order to model conditions that may arise during the life of the structure. Short-term analyses model conditions prevailing before pore water pressure dissipation occurs, such as the end-of-construction condition. Unconsolidated-undrained test parameters, which yield a relatively high cohesion value and a low or zero friction value, are appropriate for short-term analyses. Long-term analyses model conditions prevailing after shear-induced pore pressures have dissipated. For long-term analyses, consolidated-drained test parameters are appropriate. These tests usually yield a relatively high value for internal friction and a low or zero value for cohesion.

b. Passive soil pressures. If a structure is moved toward the backfill, lateral soil pressures increase and shear stresses reverse direction, first decreasing and then increasing to a maximum at failure (Figure 5-4). Full development of passive pressure requires much larger structure rotations than those required for the active case, as much as 0.02 to 0.2 radians (Figures 5-1 and 5-2). However, the rotation required to develop one-half of the passive pressure is significantly less, as little as 0.005 radians. The designer must be certain that soil on the resisting side of any structure will always remain in place and not be excavated or eroded before its effect is included in the stability analyses.

c. At-rest soil pressure. If no structural movement occurs, then the at-rest condition exists.

d. Design soil pressures - driving side. In practice, the active and passive soil pressure conditions seldom exist. Hydraulic structures are designed using conservative criteria that results in relatively stiff structures. Structures founded on rock or stiff soils usually do not yield sufficiently to develop active pressures. Even for foundations capable of yielding, experiments with granular backfill (Matsuo, Kenmochi, and Yagi 1978) indicate that following initial yield and development of active pressures, lateral pressures may in time return to greater values. Another reference (Casagrande 1973) states that the gradual buildup of the backfill in compacted lifts produces greater-than-active pressures, as do long-term effects from vibrations, water level fluctuations, and temperature changes. For these reasons and because large rotations are required for the development of passive pressures, soil pressures on both the driving side and the resisting side of the single wedge will be estimated by using the developed soil strength parameters, as defined in paragraph 5-2. These parameters are then used to calculate the *equivalent-fluid* soil pressure coefficients (K).

(1) General wedge method for equivalent fluid pressure coefficients. Lateral soil forces are assumed to act parallel to the top surface of driving side wedges when the surface slopes downward toward the structure. Equivalent fluid-pressure coefficients are calculated as follows:

$$\alpha = \tan^{-1}\left(\frac{C_1 + \sqrt{C_1^2 + 4\,C_2}}{2}\right) \tag{5-4}$$

where α = the critical slip plane angle for the soil wedge (see Appendix E for a derivation of α)

$$C_2 = \frac{t + \left(\frac{2V}{\gamma(h^2 - d_c^2)}\right)(1 + \tan^2\phi_d)\tan^2\beta + \left(\frac{2\,c_d}{\gamma(h + d_c)}\right) r}{A}$$

$$A = \tan\phi_d + \tan\delta - \left(\frac{2V}{\gamma(h^2 - d_c^2)}\right)(1 + \tan^2\phi_d) + \left(\frac{2\,c_d}{\gamma(h + d_c)}\right) r$$

$r = 1 - \tan\delta \tan\phi_d - \tan\beta\ (\tan\delta + \tan\phi_d)$

$s = \tan\beta + \tan\phi_d + \tan\delta\ (1 - \tan\beta \tan\phi_d)$

$t = \tan\phi_d - \tan\beta - (\tan\delta + \tan\beta) \tan^2\phi_d$

ϕ = soil internal friction parameter

ϕ_d = developed internal friction parameter

c = soil cohesion parameter

c_d = developed cohesion parameter

ß = top surface slope angle, positive when slope is upward when moving away from the structure (When the top surface of the backfill is broken, solutions for α may be obtained by using analogous positive and negative strip surcharges.)

δ = wall friction angle. When β is positive, $\delta = \beta$. When β is zero or negative $\delta = 0$. Vertical shear (drag), as discussed in Appendix F, shall not be used to calculate the value of α or equivalent fluid soil pressure coefficients. However, drag may be used in addition to lateral soil pressures when the requirements of Appendix F are satisfied.

γ = average unit weight of soil (moist weight above water table, buoyant weight below)

V = strip surcharge

h = height of vertical face of soil wedge

d_c = depth of cohesion crack in soil (should always be assumed filled with water when calculating lateral forces)

The equivalent fluid-pressure coefficients are:

$$K = \frac{1 - \tan\phi_d \cot\alpha}{\cos\delta\,[(1 - \tan\delta\tan\phi_d) + (\tan\phi_d + \tan\delta)\tan\alpha]}$$

and for soils that possess cohesive properties as well as internal friction:

$$K_c = \frac{1}{2\cos^2\alpha\,(\tan\alpha - \tan\beta)\,[1 - \tan\delta\tan\phi_d + (\tan\delta + \tan\phi_d)\,tanalpha\,]}$$

The equation for the depth of a cohesive crack is:

$$d_c = \frac{2K_c c}{K\gamma\left(\dfrac{\tan\alpha}{\tan\alpha - \tan\beta}\right)}$$

And the total lateral soil force is calculated as:

$$P = \frac{1}{2}K\gamma\left(\frac{\tan\alpha}{\tan\alpha - \tan\beta}\right)(h - d_c)^2 + K\,V\tan\alpha$$

Examples are presented in Appendix D.

When any of the variables in the above equations are not present in a particular problem, they are set equal to zero, thereby simplifying the equations. Figure 5-3 illustrates a wedge containing cohesionless soil, showing the methods used to calculate the lateral force, and the soil pressure at any point on the vertical face of the wedge. Figure 5-4 shows similar information for a wedge consisting of a cohesive soil. Figure 5-5 shows the method used to determine the pressure distribution for a strip surcharge (a line load V).

When ß is greater than ϕ_d a solution for α cannot be obtained from Equation 5-4 because the number under the radical will be negative, making the square root indeterminate. However, when ß is equal to ϕ_d, Equation 5-4 will give a value for α equal to ß and ϕ_d. When $\alpha = \phi_d =$ ß, the total lateral soil force, for a granular soil not supporting a strip surcharge, is:

$$P = \frac{1}{2} \gamma \, h^2 \cos \phi_d$$

This equation gives the maximum driving side lateral soil force that can occur and should be used when ß is equal to or greater than ϕ_d.

(2) *Equivalent fluid pressure coefficients from Coulomb's equation.* Coulomb's equation may be used to calculate the equivalent fluid-pressure coefficient when the surface of the backfill wedge is planar and unbroken, if certain conditions are met. These conditions are:

- When backfill has a sloping top surface. There can be only one soil material. The water-table must be either completely above or completely below the backfill. The backfill must be cohesionless. Surcharges must be uniformly distributed and cover the entire top surface of the backfill wedge.

- When backfill has a horizontal top surface. There may be more than one soil material, if the top surface of all soil layers are horizontal. The water-table may lie within the backfill. The soil may be either cohesive or cohesionless. Surcharges must be uniformly distributed and cover the entire top surface of the backfill wedge.

When any of the above conditions are not applicable, the equivalent fluid pressure coefficient determined by the wedge method shall be used. The equation for the equivalent fluid-pressure coefficient, using the Coulomb Equation is:

$$K = \frac{\cos^2 \phi_d}{\cos \delta \left[1 + \sqrt{\dfrac{\sin(\phi_d + \delta)\sin(\phi_d - \beta)}{\cos \delta \cos \beta}} \right]^2}$$

and the total lateral soil force is:

$$P = \frac{1}{2} K \gamma \, h^2$$

(3) Equivalent fluid pressure coefficients for simple conditions. When the top surface of the wedge is horizontal, planar, supports a uniform surcharge covering the entire top surface, and the soil possesses cohesive strength as well as internal friction, the equivalent fluid pressure coefficients in the preceding equations reduce to the following simple expressions:

$$K = \frac{1 - \sin \phi_d}{1 + \sin \phi_d} = \tan^2 (45° - \frac{\phi_d}{2})$$

$$K_c = \sqrt{K}$$

$$d_c = \frac{2 \, c_d}{\gamma \sqrt{K}}$$

e. Design soil pressures - resisting side. Developed soil pressures on the resisting side may also be calculated using the developed soil strength parameters. In this manual, soil pressures and forces on the resisting side are generally assumed to act horizontally (wall friction angle $\delta = 0$). The equivalent fluid-pressure coefficient for soil pressures on the resisting side is calculated as follows:

$$\alpha = \tan^{-1}\left[\frac{-C_1 + \sqrt{C_1^2 + 4C_2}}{2}\right]$$

$$C_1 = \frac{2\tan^2\phi_d - \dfrac{4V}{\gamma h^2}[\tan\beta(1+\tan^2\phi_d)] + \dfrac{4c_d}{\gamma h}(\tan\phi_d - \tan\beta)}{A}$$

$$C_2 = \frac{\tan\phi_d(1+\tan\phi_d\tan\beta) + \tan\beta + \dfrac{2c_d(1+\tan\phi_d\tan\beta)}{\gamma h} - \dfrac{2V\tan^2\beta(1+\tan^2\phi_d)}{\gamma h^2}}{A}$$

where

$$A = \tan\phi_d + \frac{2c_d(1+\tan\phi_d\tan\beta)}{\gamma h} + \frac{2V(1+\tan^2\phi_d)}{\gamma h^2}$$

Then the equivalent fluid-pressure coefficients for resisting side pressures are:

$$K_P = \frac{1+\tan\phi_d\cot\alpha}{1+\tan\beta\tan\phi_d - (\tan\phi_d - \tan\beta)\tan\alpha}$$

and when cohesion is present:

$$K_{cP} = \frac{1}{2\cos^2\alpha(\tan\alpha - \tan\beta)[1+\tan\beta\tan\phi_d - (\tan\phi_d - \tan\beta)\, tanalpha]}$$

The developed soil force on the resisting side is then:

$$P_P = \frac{1}{2}K_P\gamma\left(\frac{\tan\alpha}{\tan\alpha - \tan\beta}\right)h^2 + 2K_{cP}c_d h + K_P V\tan\alpha$$

Since cohesive cracking will not occur on the resisting side, the term for the depth of cohesive cracking (d_c) is not applicable to these equations.

5-4. Soil Pressures with Water Table Within or Above Top of Backfill Wedge

Pressures and forces due to soil and water must be calculated separately, since wall friction (δ) is not applicable to water pressure and the equivalent fluid pressure coefficients (K and K_c) are for calculating lateral soil pressure only. K for water is always equal to one. However, the effective unit weight of soil below the water table is affected by the uplift due to water. In lateral soil-pressure calculations, the moist unit weight of soil is used above the water table, and the buoyant unit weight is used below the water table. When calculating lateral water pressure and uplift, the effect of seepage (if it occurs) must be considered. Lateral soil pressures and forces are calculated as shown below, and as illustrated in Figure 5-6. These must be added to the lateral water pressure.

$$P = \frac{1}{2} p_s (h - h_s) + \frac{1}{2} (p_s + p) h_s$$

Where: P = the total lateral soil force,

and $$p_s = K \gamma_m \left(\frac{\tan \alpha}{\tan \alpha - \tan \beta} \right) (h - h_s)$$ = lateral soil pressure at water table

$$p = K \left[\gamma_m h \left(\frac{\tan \alpha}{\tan \alpha - \tan \beta} \right) - (\gamma_m - \gamma_b) h_s \right]$$ = lateral soil pressure at bottom of soil wedge

γ_m = moist unit weight of soil, use above water table

γ_b = buoyant unit of weight of soil, use below water table

h = height of vertical face of soil wedge

h_s = height of water table above bottom of wedge

5-5. Earthquake Inertial Forces on Structures

The seismic response of structures with backfill is complicated because in addition to the interaction with the foundation, the structure is also subjected to the dynamic soil pressures induced by ground shaking. Dynamic backfill pressures are related to the relative movement between the soil and the structure and the stiffness of the backfill. Behavior of the structure may be controlled by rocking and/or translation in response to earthquake shaking. The type of response will correlate to different distributions of backfill pressure acting against the wall. The appropriate method for analyzing the backfill pressure may be categorized according to the expected movement of the backfill and structure during seismic events.

a. Behavior of Backfills.

(1) Non-Yielding Backfills. For low intensity ground motions the backfill material may respond within the range of linear elastic deformations. Walls with non-yielding backfills can be expected to have dynamic soil pressures greater than those predicted by the Mononobe-Okabe method. The dynamic soil pressures and associated forces in the backfill may be analyzed as an elastic response using Wood's method as described in ITL 92-11 (Ebeling and Morrision, 1992). A reasonable estimate for determining the additional lateral seismic soil force against a soil retaining structure, for non-yielding backfill conditions, can be determined as

$$F_{sr} = \gamma h^2 k_h$$

where: F_{sr} = Lateral seismic force representing dynamic soil pressure effects

γ = Unit weight of soil. Use moist or submerged unit weight when all soil is above or below the water table. For partially submerged soils the unit weight shall be proportioned by a weighted average.

h = Height of backfill

k_h = Effective peak ground acceleration, expressed as a decimal fraction of the acceleration of gravity

The seismic component of the total soil force F_{sr} is assumed to act at a distance of 0.63 h above the base of the wall. This force must be combined with the structure lateral inertial force, and if water is present, hydrodynamic seismic forces to obtain the total seismic force on the wall. Evaluation of a wall with non-yielding backfill for the aforementioned seismic forces is illustrated by Example 32 of ITL-92-11 (Ebeling and Morrison, 1992). The various seismic forces described above must be combined with static soil pressure forces and static water pressure forces to get the total force on the wall. Soil retaining structures not meeting stability criteria using the preliminary screening method should be evaluated using refined analysis techniques described in ITL-92-11 (Ebeling and Morrison, 1992).

(2) Yielding Backfills. The relative motion of the structure and backfill material may be large enough to induce a limit or failure state in the soil. This condition may be modeled by the Mononobe-Okabe method (Mononobe and Matsou (1929), and Okabe (1926)), in which a wedge of soil bounded by the structure and an assumed failure plane are considered to move as a rigid body with the same horizontal acceleration. The dynamic soil pressures using this method are described in Appendix G.

(3) Partially Yielding Backfills. The intermediate condition in which the backfill soil undergoes limited nonlinear deformations corresponds to the shear strength of the soil being partially mobilized. The dynamic backfill pressures may be estimated using an idealized constant parameter, SDOF model of a semi-infinite uniform soil layer (Veletsos and Younan 1994) or a frequency-independent, lumped parameter, MDOF system. The dynamic pressures for an irregular backfill may be analyzed using a soil-structure-interaction model such as FLUSH (Lysmer et al. 1975). The wall is usually modeled with 2-D elements. The foundation rock is represented by 2-D plane-strain elements with an appropriate modulus, Poisson's ratio, and unit weight. Transmitting boundaries in the form of dashpots are introduced at the sides of the foundation rock to account for the material nonlinear behavior with depth. The shear modulus and soil damping vary with the level of shearing strain, and this nonlinear behavior is usually approximated by an equivalent linear method. The boundary conditions for the backfill may also be represented by dashpots. Hydrodynamic pressures exerted on the wall are computed using the Westergaard formula.

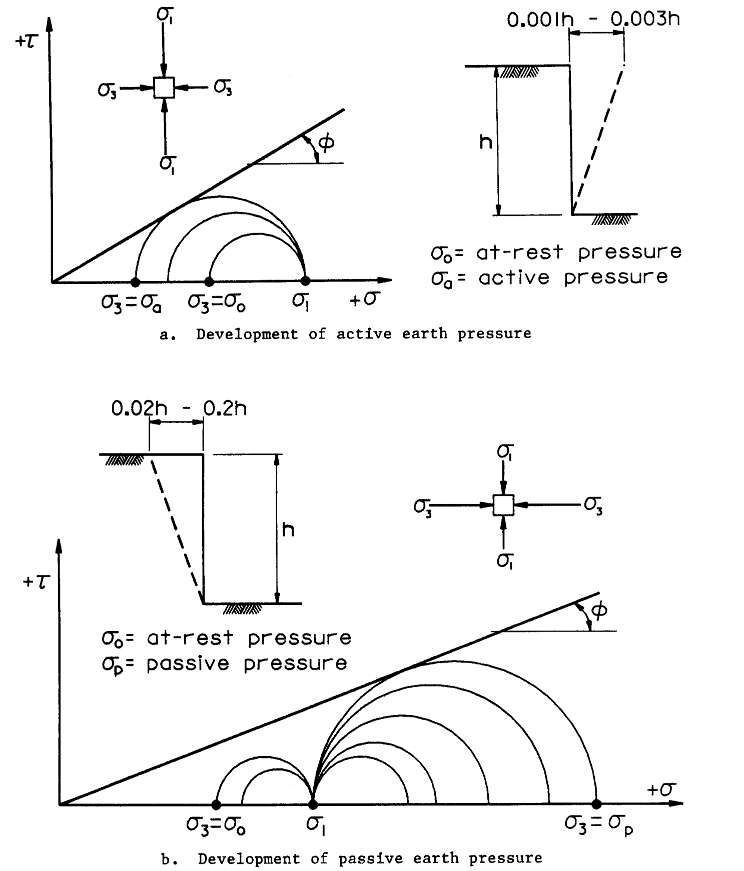
b. Simplified Wedge Method. A seismic coefficient method may be used to estimate the backfill and wall inertial forces acting on a single wedge. Theoretically, a structure (wedge) may behave as a rigid body that is fully constrained along its base and sides by the ground, so all parts of the wedge would be uniformly affected by accelerations which are identical to the time history of the ground motions. Therefore it would be appropriate to use a seismic coefficient equal to the peak ground acceleration for stability analysis of short, stiff structures. However, field and test data show that most structures do not behave as a rigid body, but respond as a deformable body subjected to effective ground motions. Thus the magnitude of the accelerations in a deformable wall may be different than those at the ground surface, depending on the natural period and damping characteristics of the structure and the shaking characteristics of the ground motions. Furthermore the maximum acceleration will only affect the structure for a short interval of time, and the inertia forces will not be equivalent to those of an equal static force which would act for an unlimited time, so the deformations resulting from the maximum acceleration will be smaller. Design or evaluation of structures for zero relative displacement under peak ground accelerations is unrealistic, so the seismic stability analysis should be based on a seismic coefficient, which recognizes that an acceptably small amount of lateral displacement will likely occur during a major earthquake. Experience has shown that a seismic coefficient equal to 2/3 of the peak ground acceleration is a reasonable estimate for many hydraulic structures. For partially yielding backfill, the strength mobilization factor should be equal to the reciprocal of the minimum required sliding safety factor for that load case. More information about the simplified wedge method is included in appendix G.

5-6. Mandatory Requirements

For a general discussion on mandatory requirements see Paragraph 1-5. As stated in that paragraph, certain requirements within this manual are mandatory. The following are mandatory for Chapter 5.

a. Developed Soil Strength Parameters. Lateral soil forces acting on a single wedge shall be determined using developed soil strength parameters as described in paragraph 5-2.b.

b. Sliding Factor of Safety. The factor of safety for sliding shall be calculated as defined in paragraph 5-2.c.



a. Development of active earth pressure

b. Development of passive earth pressure

Figure 5-1 Soil Pressures for Cohesionless Material

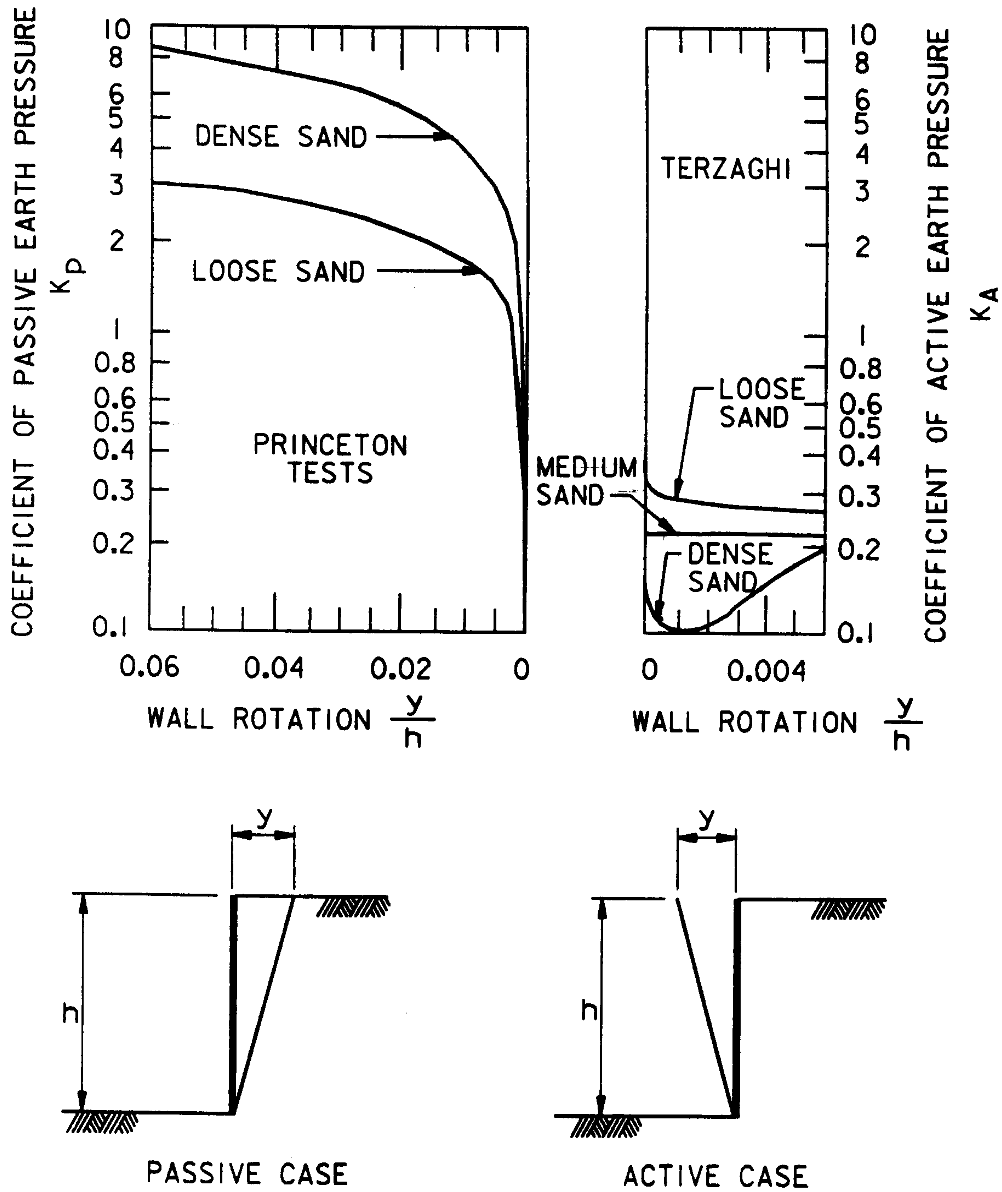
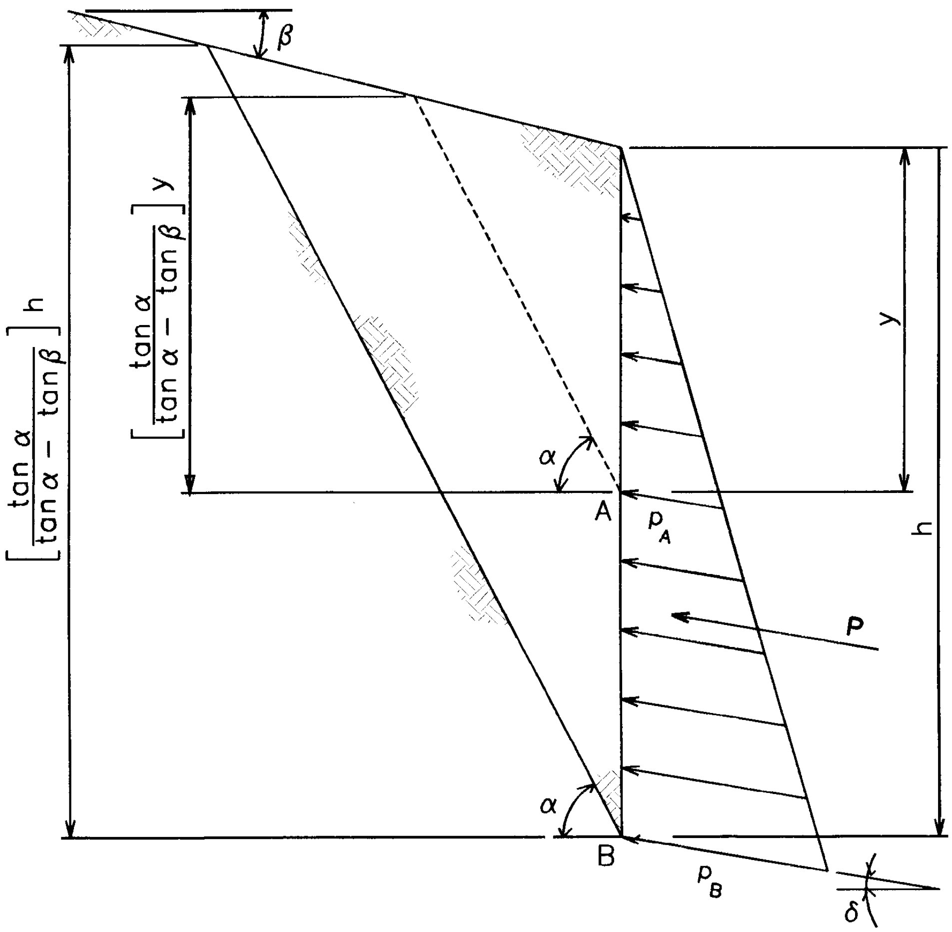


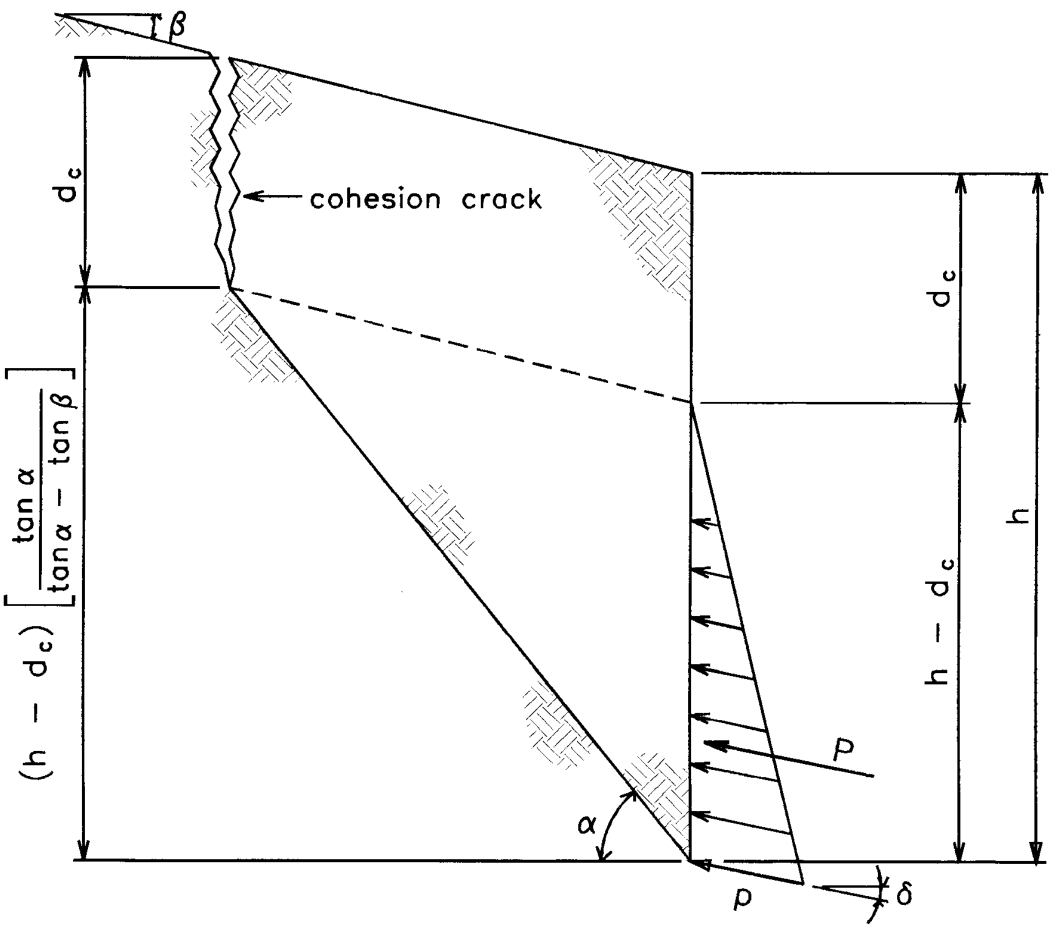
Figure 5-2 Soil Pressures versus Wall Movements



$$p_A = K\gamma\left[\frac{\tan\alpha}{\tan\alpha - \tan\beta}\right]y \qquad p_B = K\gamma\left[\frac{\tan\alpha}{\tan\alpha - \tan\beta}\right]h$$

$$P = 1/2\, K\gamma\left[\frac{\tan\alpha}{\tan\alpha - \tan\beta}\right]h^2$$

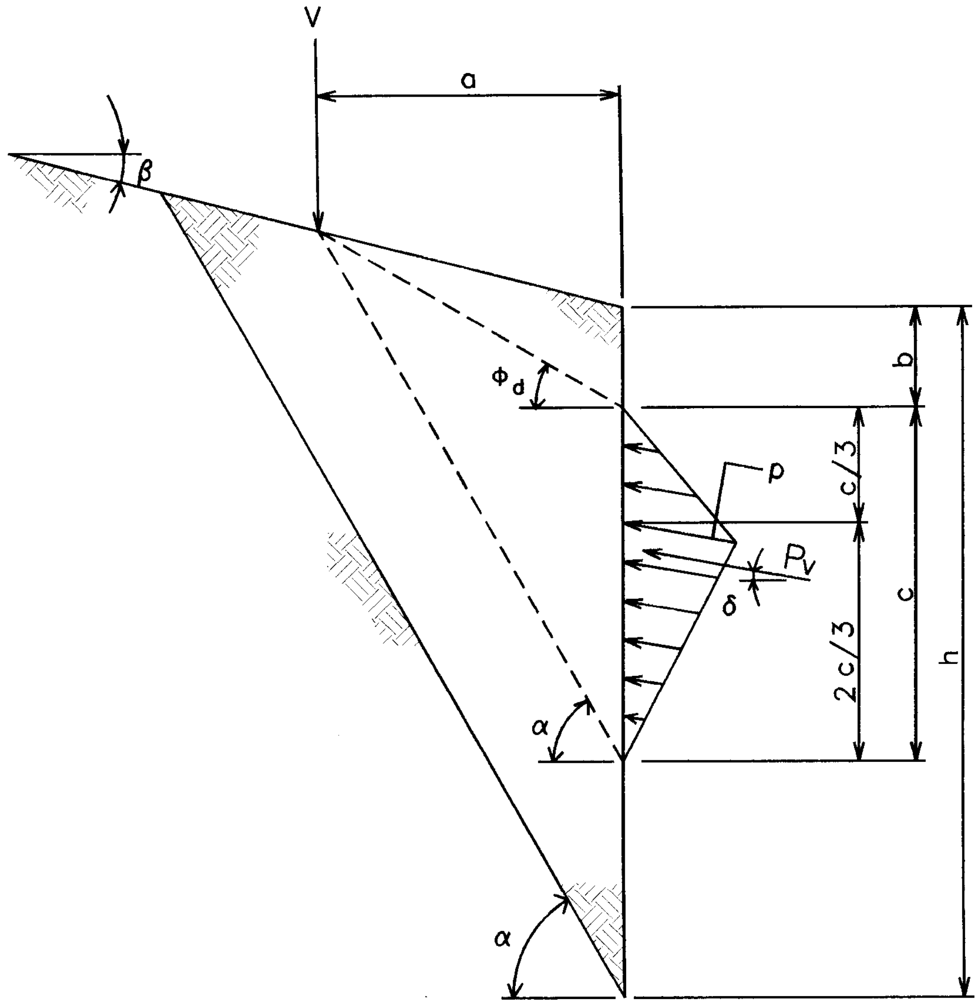
Figure 5-3 Calculation of Soil Pressures for Cohesionless Material



$$p = K\gamma\left[\frac{\tan\alpha}{\tan\alpha - \tan\beta}\right](h - d_c)$$

$$P = 1/2\ K\gamma\left[\frac{\tan\alpha}{\tan\alpha - \tan\beta}\right](h - d_c)^2$$

Figure 5-4 Calculation of Soil Pressures for Cohesive Material



$$b = a\,(\tan\Phi_d - \tan\beta)$$

$$c = a\,(\tan\alpha - \tan\Phi_d)$$

$$P_v = K\ V\ \tan\alpha$$

$$p = \frac{2P_v}{c}$$

Figure 5-5 Lateral soil pressures due to strip surcharge loads

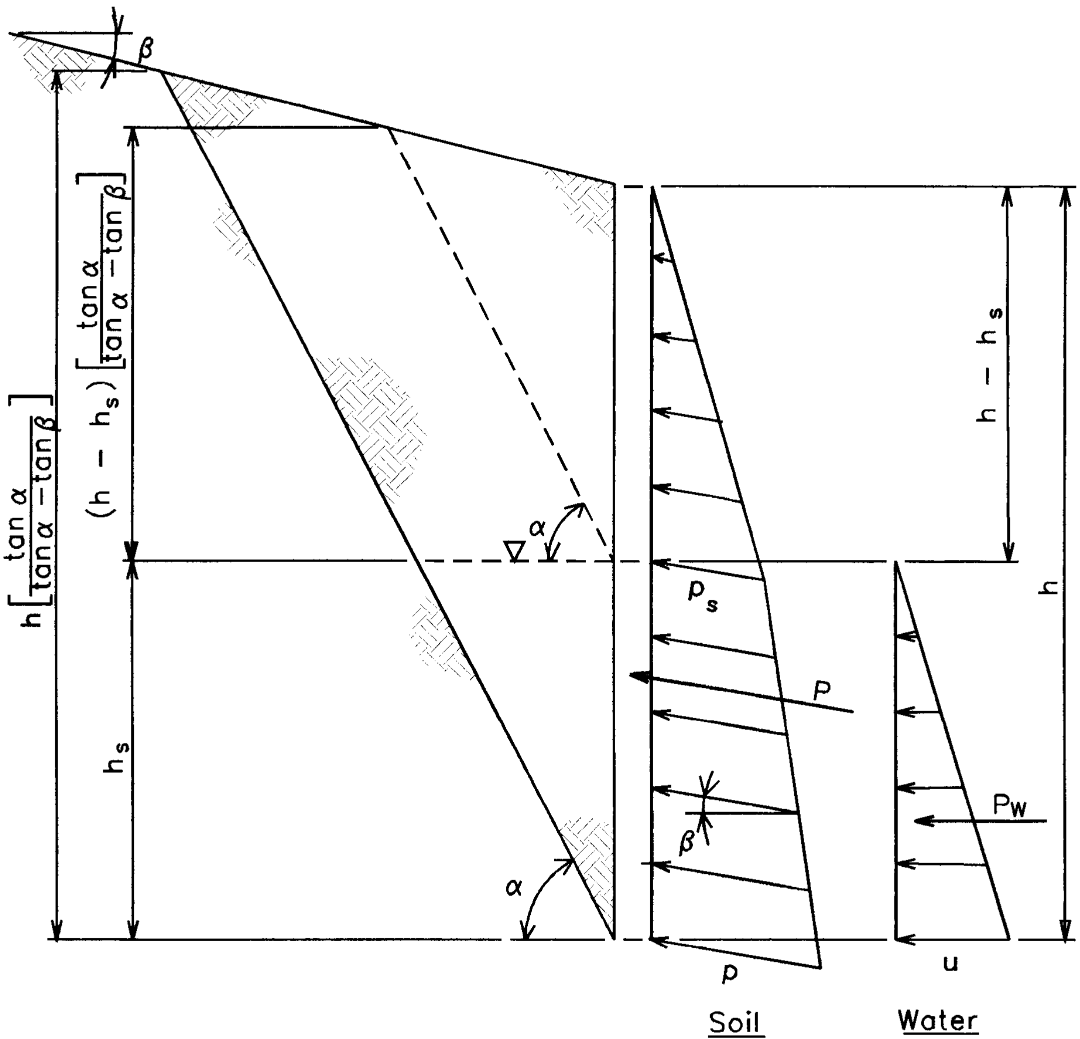


Figure 5-6 Lateral Soil Pressures with Water Table

Chapter 6
Stability Considerations and Analytical Methods

6-1. General

Both new and existing structures are constantly being examined to determine if they meet stability criteria. The traditional procedure is to evaluate sliding stability of these structures using the limit equilibrium method of analysis and to evaluate rotational stability using rigid body assumptions and bearing pressure distributions that vary linearly across the base. Soil and uplift loads on the structure are often based on simplifying assumptions. In the limit equilibrium method, only the stress state at failure is considered, usually as represented by the Mohr-Coulomb limit-state criterion. The simplified loads and the Mohr-Coulomb limit-state criterion are then used to obtain a single overall factor of safety against a sliding stability failure. This traditional approach is appropriate for many structures where it is difficult to predict just how a particular failure mechanism may develop. However, for existing structures where more is known about service state conditions and uplift pressures, it is possible to more precisely predict what stress changes must occur to develop a failure condition. The margin of safety can be more accurately determined when the path to failure, from initial stress conditions to limit state conditions, is known. This path to failure can be investigated using linear elastic and nonlinear finite-element numerical solutions and fracture mechanics concepts. Because the deformation of the structure and its foundation is considered in finite element analyses, and because the path to failure can be more realistically characterized through fracture mechanics, a more accurate assessment of safety can often be made using advanced analytical methods.

6-2. Traditional Methods

Traditional two-dimensional and three-dimensional, simplified, conservative stability analyses are used as the first step of a stability evaluation. Advanced analytical methods may be used to further investigate stability when the traditional methods indicate the structure is inadequate or marginal with respect to sliding stability or rotational stability.

a. Two-dimensional analysis. Two-dimensional stability analyses using traditional methods are suitable for the intermediate monoliths of structures that have uniform geometry and loading for their entire length and where the loads and resistance due to end effects are not transmitted across monolith joints. Many walls have variable height or loads along their length. Each monolith of such walls should be proportioned for stability at the low-end monolith joint, at the midpoint between monolith joints, and at the high-end monolith joint; the dimensions at different points along the wall should vary linearly between the sections for which the stability is checked.

b. Three-dimensional analysis. Some structures can not be suitably idealized as a two-dimensional structural system and therefore require a three-dimensional analysis. Examples of such structures include: nonoverflow monoliths at the ends of gravity dams, navigation-lock miter-gate monoliths, arch dams, and other structures having nonuniform geometry and/or loading. These types of structures, however, may still be analyzed using traditional methods with the following additional considerations.

(1) End effects. Figure 6-1 shows a plan and elevation of a short wall and the earth wedges that would be associated with sliding failure. However, frictional drag forces will exist on the active (driving) wedge faces abc and a'b'c', and on the passive (resisting) wedge faces edf and e'd'f' as well on the embedded end areas of the wall. These forces are generated by at-rest soil pressure on the end faces of the wedges and the wall, acting in conjunction with the internal soil friction. The frictional drag forces may be added to the numerator of Equation 5-1. When this resistance is used in conjunction with a multiple-wedge sliding analysis, as discussed in Chapter 2, $Ø_d$ (the developed value) should be used in lieu of Ø in calculations.

(2) Arch dams. Unlike a gravity dam, where stability is provided by the weight of the structural wedge, an arch dam's stability depends, not only on weight but to a much larger extent, upon arch action that transmits the imposed loads to the valley walls. Therefore, an evaluation of the foundation characteristics is required prior to laying out the dam. Following this procedure, the dam is designed so that the loads transmitted to the foundation will be within acceptable limits. In special instances artificial abutments (thrust blocks) may be used to increase the ability of the valley walls to resist these thrusts. Arch dams are constructed using monoliths in the same manner as

for gravity dams. The construction blocks or monoliths of double curvature arch dams may have a tendency to rotate upstream prior to completion. This tendency should be evaluated and a system of support should be evaluated, designed and included in the construction documents. The stability of these vertically cantilevered monoliths must be maintained at all stages of construction. EM 1110-2-2201 provides detailed information on site selection, stability requirements, design and construction criteria and procedures, methods of static and dynamic analysis, temperature studies, concrete testing requirements, foundation investigation, and instrumentation for use in the design of arch dams.

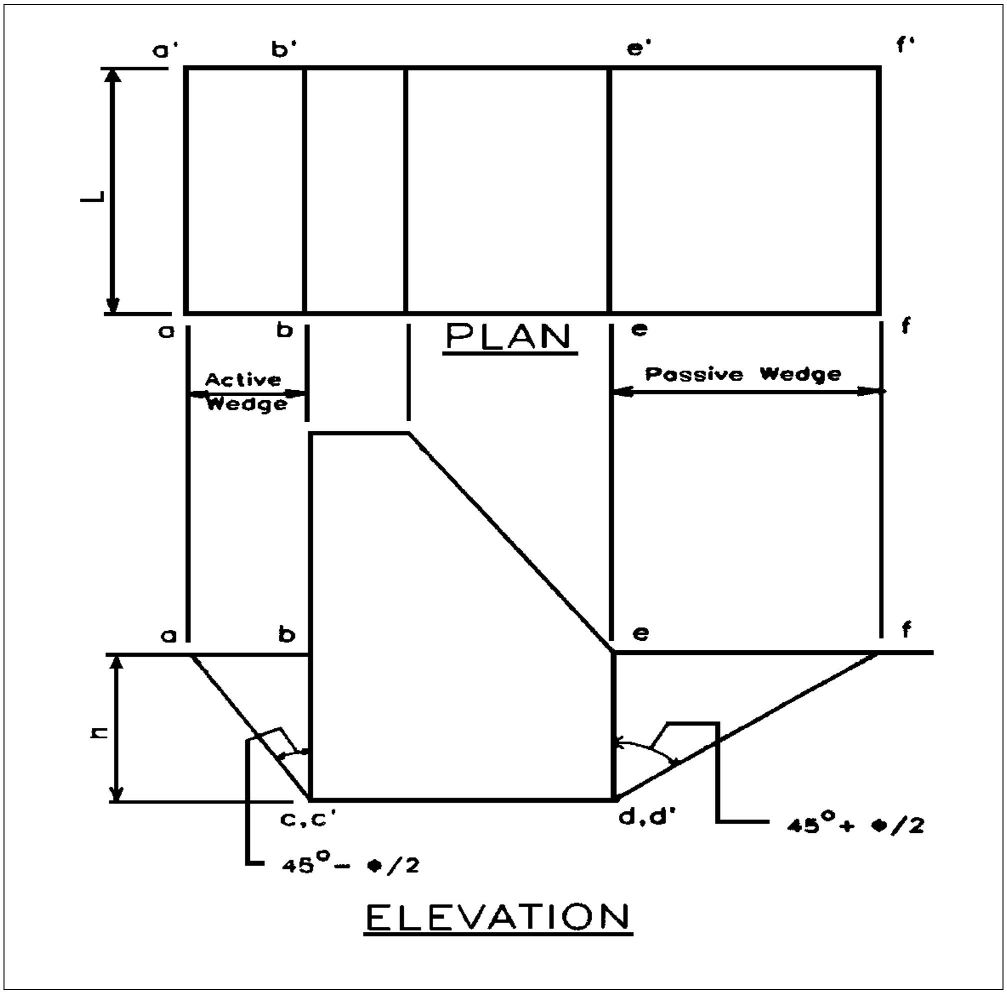


Figure 6-1. End effects on short wall

6-3. Advanced Analytical Analyses

Advanced analytical analyses can be used to gain a better understanding of structural behavior. However, the results obtained from such an analysis must be used in conjunction with the traditional analysis methods. The advanced analytical methods briefly described in this manual can be extremely difficult to perform and evaluate. Therefore, advanced analytical analyses should be conducted in conjunction with experts who can oversee the numerical modeling and can help with the interpretation of the results.

a. Stress/strain compatibility. Foundations may contain more than one material or may be made up of a combination of intact rock, jointed rock, or sheared rock. The stress-strain characteristics of these materials may be quite different. They might exhibit elastic-plastic behavior where peak shear strengths will occur simultaneously (Figure 6-2) or one or more might exhibit strain-softening behavior where only the residual strength of that material will be available at the strain level associated with peak shear in the other material (Figure 6-3). The overall strength of the foundation will depend on the stress-strain characteristics and compatibility of the various materials.

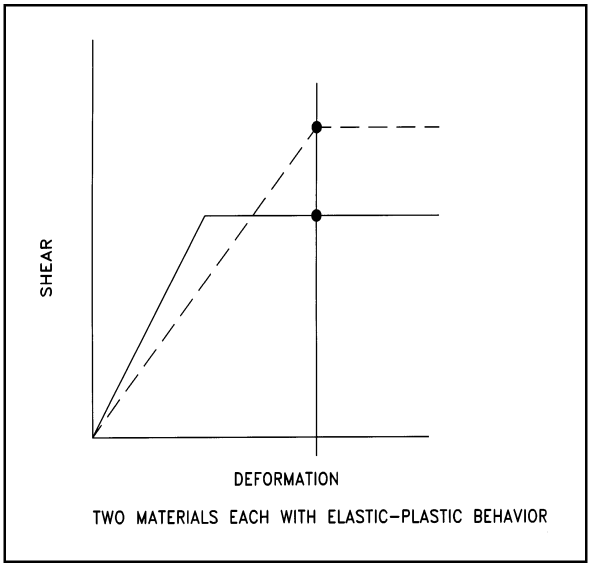


Figure 6-2

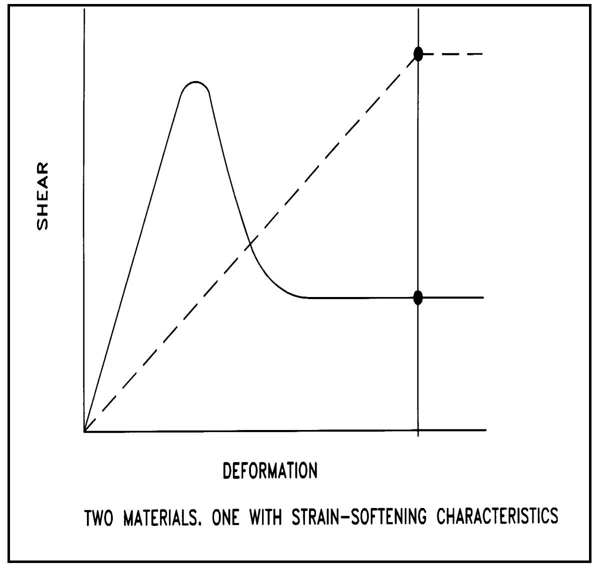


Figure 6-3

b. Deformations. The sliding stability analysis procedures described in Chapter 2 are based on a limit state approach, which satisfies only some of the equilibrium conditions and only accounts for deformations indirectly. No attempt has been made in this approach to relate shear stresses to displacements. The magnitude of displacement required to reach a limit state needs to be considered when selecting strength parameters. It is not possible, for instance, to develop peak shear and cohesive strength as well as passive resistance, all at the same strain level. Therefore, the analysis may require the use of residual shear strengths rather than peak shear strength values. In addition, the deformations associated with shear resistance may be unacceptable for reasons of serviceability. Controls on maximum displacement, needed to ensure proper function and safety, may govern over factor of safety requirements.

c. Special Circumstances. The minimum factors of safety for sliding stability are based on Mohr-Coulomb limit-state failure criterion without consideration of the deformations that occur in the structure and its foundation, or of the type of failure path the structure will experience in moving from a service-state from a limit-state condition. In many cases, it can be demonstrated by the use of finite element analysis and fracture mechanics methods that the factor of safety is substantially different from that predicted by the limit equilibrium method. Under special circumstances, such as in the case of an existing structure where the limit equilibrium method indicates that remedial action is required to improve stability, additional stability analyses may be performed using finite element and fracture mechanics procedures to verify whether stability remediation is actually required.

d. Finite element method (FEM). Finite element methods can be used to determine the manner in which loads and resistance are developed as a function of the stiffness of the foundation, stiffness of the structure, and the structure-to-foundation interface. They can also be used to calculate displacements and stresses due to incremental construction and/or load applications, and can model nonlinear stress-strain material behavior. The FEM program SOILSTRUCT, a two-dimensional plane strain-analysis program, is commonly used for soil-structure interaction problems and has been modified for use in evaluating the loss of contact along a crack, or along the structure-foundation interface. The modified version uses a procedure called the Alpha Method (Ebeling et. a1 1992) in an incremental analysis to determine the extent of cracking that might occur at the structure-foundation interface. Finite element analyses can also be used to capture soil-structure interaction effects and to investigate crack propagation using fracture mechanics techniques. Soil-structure interaction. When a structure is supported by rock or soil, a finite element analysis can provide lateral pressures and bearing pressures that are realistic since the elastic and plastic properties of the structure, foundation, and soil are all considered.

e. Linear elastic fracture mechanics (LEFM) analysis. LEFM methods are FEM-based procedures for modeling crack development in a structure or along the base of a structure. The analysis can be performed using discrete crack analysis theory, or smeared crack analysis theory. The use of LEFM analysis in the evaluation of stability is described in various engineering papers (Dewey, Reich, and Saouma 1994), (Saouma, Bruhwiler, and Boggs 1990), (Bazant 1990) and (Ebeling, Morrison, and Mosher 1996).

6-4. Computer Programs

Following is a listing and brief description of computer programs that may serve as aids when determining the stability requirements for new or existing structures. Other commercially available computer programs which are not included in the following list, such as ADINA, ANSYS, CRISP, NASTRAN, or PLAXIS may also help when performing complex stability analysis.

- 3DSAD is a U.S. Army Corps of Engineers developed program (library number X8100) that allows the user to describe the geometry of a three-dimensional structure, interactively plot the described structure, and compute weight and centroid information for individual pieces or the sum total of the structure.

- CFRAG is a U.S. Army Corps of Engineers developed program (I0018) that allows the user to analyze groundwater flow using the method of fragments. The program can be used to compute: (a) seepage through soil mediums that can be modeled using fragment, (b) head losses, (c) exit gradients, and (d) resultant uplift and lateral forces.

- CSLIDE is a U.S. Army Corps of Engineers developed program (X0075) that allows the user to assess the sliding stability of concrete structures using the limit equilibrium method described in Chapter 2.

- CSEEP is a U.S. Army Corps of Engineers developed program (X8202) that allows the user to (a) interactively generate a finite element grid, (b) perform a finite element method seepage analysis, and (c) to plot the results.

- MERLIN is an Electric Power Research Institute (EPRI) developed program that allows the user to solve problems in elasticity, plasticity, linear and nonlinear fracture mechanics, steady-state and linear-transient heat transfer, and transient seepage flow. This program includes numerous capabilities for performing fracture mechanics analyses using the discrete crack approach. Pre- and post-processing capabilities are contained in PreMERLIN and PostMERLIN, respectively.

- SOILSTRUCT is a U.S. Army Corps of Engineers developed program that allows the user to perform a two-dimensional plane-strain finite element analysis of incremental construction, soil-to-structure interaction analysis of earth-retaining structures. The program is capable of simulating incremental construction including embankment construction or backfilling, the placement of layer(s) of a reinforcement material during backfilling, dewatering, excavation, installation of a strut or tie-back anchor support system, removal of the same system, and the placement of concrete or other construction materials. The program is also capable of modeling the interface region between the soil backfill and the structure using interface elements.

- CG-DAMS is an EPRI developed program that is a specialized nonlinear, finite element, concrete gravity-dam stability code that predicts cracking potential under all types of loads including extreme flood and earthquake loads. The code has library models of typical dam and foundation cross-sections that can be customized by entering elevations and slopes of the actual dam. Uniquely shaped dam profiles are modeled by user input. Output routines provide the sliding stability factor of safety, structural deformations, stress and pressure profiles in the dam foundation, and crack length.

6-5 Mandatory Requirements

There are no mandatory requirements in this chapter.

Chapter 7
Evaluating and Improving Stability of Existing Structures

7-1. General

The stability of existing structures is sometimes reevaluated due to project modifications, changes in site conditions, improved knowledge of site data, or changes in stability criteria (ER 1110-2-1155). Since modifications to improve stability are often expensive, each structure should undergo a systematic, phased evaluation process to determine whether remediation is necessary. To avoid unnecessary modifications, all types of resisting effects, including those described below, should be considered. These include vertical friction, side friction, or three-dimensional effects. It may not be necessary to modify an existing structure that does not satisfy the requirements for new structures, especially when the structure's remaining life is short (scheduled to be replaced or removed) or when there are no indications of any stability problem. Where current stability requirements cannot be satisfied, waivers may be obtained subject to approval by CECW-E.

7-2. Procedures

The following procedures are to be used in the evaluation of existing structures and in the determination of necessary corrective measures to improve stability. The procedures should be considered as guides and are not intended to replace engineering judgment by the project engineers. The stability condition should always be reviewed when there are significant changes in the loading conditions, severe damage due to accident, aging or deterioration, discovery of structural deficiencies, revisions of stability criteria to become more conservative, or when required by ER 1110-2-100. The phases listed below shall be followed in sequence until the prescribed performance objectives described in Chapter 3 are satisfied. Existing structures can qualify for use of the *well-defined* site information category. This will permit use of lower factors of safety when evaluating sliding stability.

a. PHASE I; Preliminary analysis and evaluation. Preliminary analysis should be performed based upon available data and actual conditions of the structure. Before performing the analysis, collect and review all the available data and information for the structure including geologic and foundation data, design plans, as-built plans, periodic inspection reports, damage reports, plans of previous modifications to the structure, measurements of movement, instrumentation data, and other pertinent information. It may be necessary for the engineer to inspect and examine the existing structure to assess its condition. The first step in evaluating the stability of an existing structure is to perform a preliminary two-dimensional stability analysis. Uplift pressures, the angle of internal friction, and cohesion should be evaluated using parametric studies to assess the impact of each parameter on the factors of safety. If the results of the Phase I preliminary analysis indicate that the structure meets the safety and performance objectives of Chapter 3, no further investigations are required. Otherwise, various options to meet safety and performance objectives should be investigated.

b. PHASE II; Study, investigation, and comprehensive analyses. When the preliminary analysis indicates the structure does not meet safety and performance objectives, a plan of action for a comprehensive stability evaluation should be developed. The plan should determine the extent of the explorations and testing program needed to accurately define foundation strength parameters, the analytical program needed to accurately define the loading conditions, the remedial schemes to be studied, and the extent of any additional parametric studies. An exploration, sampling, testing, and instrumentation program should be developed, if needed, to determine the magnitude and the reasonable range of variation for the parameters which have significant effects on the safety and performance of the structure as determined by parametric studies. Testing laboratories should be used to the maximum extent practicable to develop shear strength data for the foundation and backfill materials. Comprehensive stability analyses should be performed using the material properties and strength information obtained from the sampling and testing program. Consideration may be given to reducing backfill pressures to active-state conditions provided the displacements required to reach an active-state condition meet the performance objectives in Chapter 3. Remedial measures will not be required to improve stability should the parametric studies conducted in Phase II indicate the safety and performance objectives of Chapter 3 can be met without remediation.

c. PHASE III; Advanced analytical studies and reliability analyses.

(1) Advanced analytical studies. If the stability of the structure is still in question after completing preliminary and comprehensive analyses, advanced analytical studies should be performed. These studies should use two- and three-dimensional finite element methods to capture the interaction between the foundation, backfill, and the structure, and to capture the capacity of the structural system to distribute loads to adjacent monoliths and abutments. A fracture mechanics analysis (Dewey, Reich, and Saouma, 1994) may be necessary to evaluate cracking and the progressive development of uplift pressures along potential failure planes. However, nonlinear analyses are not simple to perform, and fracture mechanics computer programs are not readily available. Fracture mechanics analysis should only be undertaken in special circumstances.

(2) Reliability analyses. Reliability analyses provide a alternate means of evaluating the stability of existing structures. This process combines what is known about a structure, with reasonable limits for the unknowns, to aid the engineer in assessing in probabilistic terms the potential for a stability failure. Although generally used for making investment decisions about projects competing for major rehabilitation funds, the process can be used to evaluate the risks associated with structures that, when analyzed by conventional deterministic procedures, indicate a stability-failure potential.

(3) Vertical shear. Many existing gravity earth-retaining structures have performed satisfactorily for many years although they do not meet stability criteria. Several research programs have addressed the stability of existing structures. The results are provided by Ebeling, Duncan, and Clough (1990), Filz and Duncan (1992), and Ebeling, Pace, and Morrison, (1997). This research demonstrated that, for rigid gravity walls on rock, where the rotation of the walls is not sufficient to fully mobilize the shear resistance of the backfill, a vertical shear force (drag) will develop on the back of the wall as a result of vertical settlement within the backfill. This shear force helps stabilize gravity walls founded on rock and should be considered when evaluating existing structures. The vertical shear force can be determined by a soil-structure interaction analysis using nonlinear finite element programs such as SOILSTRUCT as part of the Phase III analysis. Additional information and details on the use of vertical shear in stability analysis is presented in Appendix F.

7-3. Improving Stability

Stability can be improved through several means such as by the use of foundation drains to reduce uplift, by adding anchors, and by adding concrete mass, concrete buttresses, or buttressing embankments.

a. Reducing uplift pressures. In most cases, the pressure reduction provided by foundation drains is much greater than what is usually assumed for design. However, experience has shown that drains can clog, and this sometimes is temporarily unnoticed. If drains are properly maintained and if foundation testing and flow analysis provide supporting justification, the drain effectiveness can be increased beyond 50 percent subject to approval from CECW-E. This criterion will depend on the pool operational plan, instrumentation to verify and evaluate uplift assumptions, and an adequate drain maintenance program. When actual uplift pressures are known, these uplift pressures can be extrapolated to pool levels used for the stability evaluation. However, extrapolation of existing uplift readings at low-pool elevations for use under high-pool conditions must be done with caution and with full consideration of the effect joint aperture changes can have on high-pool uplift pressures and their distribution from upstream to downstream within the foundation. Joints in the foundation rock can open or close as pool conditions change and the joint aperture change may not be uniform. Also, joint aperture changes can result in something other than linear uplift pressure distribution from upstream to downstream within the foundation. For these reasons, the use of existing uplift pressures in stability analyses must be supported with uplift pressure readings that reflect uplift pressure changes that have occurred under various pool levels and various climatic conditions. Information on the influence that geology, foundation treatment, drainage, changes in reservoir elevation, and seasonal temperature variations have on uplift pressures can be found in EPRI (1992). Flood-pool conditions that cause tension at the heel of the structure must be checked to determine if a crack will develop between the structure and foundation, resulting in full uplift pressures in the crack. Uplift records must be sufficient to adequately define distributions across the base from headwater to tailwater, and cover as many years as necessary to discern if there is a potential for any uplift pressure changes with time, and to provide information under various pool conditions. In existing gravity dams and other structures with drainage systems, the most economical way of improving stability is usually to clean out existing drains or to add new foundation drains. The effectiveness of new drains however, will depend on the nature of the foundation, and upon whether or not the new drains intersect pressurized discontinuities. For structures without drainage systems, the uplift pressures can be reduced through the use of impervious blankets and cutoffs, although these methods of uplift reduction are much less reliable. Ongoing monitoring is required for drains, impervious blankets, and cutoff systems to ensure actual uplift pressures are within the range of values assumed for stability evaluation.

b. Structural anchor systems. Tensioned vertical or inclined anchors can be installed to improve sliding stability. Vertical anchors installed across potential failure planes improve the sliding resistance by adding a normal force that increases the shear-friction component of the sliding shear resistance. Inclining the anchors in the upstream direction will provide a horizontal component that can be used for additional shear resistance. However, untensioned anchors should not be used to improve sliding or rotational stability. Anchor systems are covered in detail in Chapter 8.

c. Adding concrete and buttressing to improve stability. Adding weight to a structure will increase compressive stress on failure planes and, thereby, increase the shear-frictional resistance to sliding. Concrete can be added to the front face, back face, or on top of the structure to increase the weight. The structure can also be buttressed with concrete blocks or with soil to improve sliding resistance. Often concrete blocks are placed against the toe of the structure and anchored with stressed tendons or bars (tensioned structural anchors) to further increase sliding resistance.

7-4. Case Histories

a. Eisenhower and Snell locks. Eisenhower and Snell locks are located on the St. Lawrence Seaway. The Corps of Engineers designed and constructed the two locks for the St. Lawrence Seaway Commission in the 1950s. Since that time, Corps personnel participated in annual inspections of the two locks. Cracking had been observed in the lock monoliths. Advanced analytical studies were required to determine the adequacy of the existing lock monoliths in terms of strength and stability, and to determine the advisability of lock monolith rehabilitation. A nonlinear finite element analysis, where loads are applied incrementally to simulate the actual field conditions, was used to evaluate monolith performance. Six categories were modeled in the FEM analysis:

- Initial conditions after excavation.
- Placement of the lock wall.
- Raising of the water table in the backfill.
- Formation of monolith cracks.
- Placement of tensioned structural anchors within the monoliths.
- Operation of the lock after rehabilitation.

The refined analysis procedure demonstrated the lock monoliths, once rehabilitated by installing tensioned structural anchors within the monoliths, could operate safely without structural or stability problems. This could not have been accomplished using traditional stability analysis procedures. (Mosher, Bevins, and Neeley 1991).

b. Lock 27 study. Lock 27 is located on the Mississippi River. Monolith 7E is a gravity earth-retaining structure that when evaluated using conventional equilibrium-based analysis methods demonstrated there was crack propagation at the structure-foundation interface. The crack propagation was evaluated using a finite element analysis procedure, a fracture-mechanics discrete crack analysis procedure, and a fracture-mechanics smeared crack analysis procedure. The finite element analysis utilized the SOILSTRUCT- Alpha procedure described in Chapter 6 above. The special-purpose FEM code MERLIN was used for the discrete crack analysis procedure, and the special-purpose FEM code CG-DAMS was used for the smeared crack analysis procedure. The crack estimated propagation length at the base of Monolith 7E was similar in each of the three advanced analytical methods and significantly less than that predicted by the conventional equilibrium-based analysis method. Base pressure distribution was also significantly different from the linear distribution assumed for the traditional method of analysis (Ebeling 1996).

c. Stewart Mountain Dam. Stewart Mountain Dam is a thin arch dam built in 1929 to 1930. An evaluation of the dam revealed that it was not dynamically stable during a Maximum Credible Earthquake (MCE) event; therefore, to stabilize the dam, 62 tensioned, structural anchors on 2.6m (8.5ft) to 3.0m (10.0ft) center-to-center spacing were installed from the crest of the dam and generally extended into the foundation. The bearing plates were designed to withstand a 4,450kN (1,000k)-tension load. A new overlay of 35mP (5,000psi) compressive strength concrete was placed on the crest of the dam to withstand punching shear. This overlay was designed to act as a beam on a flexible foundation because of the differences in the modulus of elasticity of the new overlay concrete and that of the existing dam concrete. A two-dimensional ADINA finite-element study was performed on a typical localized cross section of the top of the dam. The bearing plate, overlay, and existing dam concrete were modeled. Reinforcing for the overlay was designed to withstand the computed horizontal tension stresses under the bearing plate and the shear stresses below the edges of the bearing plate. For a detailed discussion of the design, refer to Bureau of Reclamation 1991.

7-5 Mandatory Requirements

There are no mandatory requirements in this chapter.

Chapter 8
Anchoring Structures

8-1. General

Structural anchors are often used to improve the stability of existing structures but, generally, should not be used as a primary means to stabilize new, large mass concrete structures. If a situation exists where anchors are used to stabilize a new structure (space limitations and/or economics dictate their use), prior approval from CECW-E must be obtained. This chapter provides guidance on the design of anchoring structures to rock, it does not include information on soil anchors. Use of soil anchors for permanent structures is not allowed without prior approval from CECW-E. Untensioned structural anchors are commonly used in designs of new structures to stabilize thin concrete members on or against competent rock. Thin concrete members include walls, chute slabs, stilling basin slabs, and paved channel slabs. For such uses, prior approval from CECW-E is not required. Following are brief descriptions of new projects, which did utilize anchors into rock to stabilize large structures.

a. Bay Springs Lock. This lock was located in a deep-rock excavation so a considerable savings was achieved by anchoring the lock walls to the rock. This reduced the amount of rock excavation and the amount of mass concrete. This project was designed by the Nashville District.

b. Bonneville Lock. This lock and the upstream approach channel wall are in close proximity to a railroad. So, to avoid relocating the railroad it was determined the most economical design was to use structural anchors to stabilize the walls. This project was designed by the Portland District.

c. Wanapum Dam. Tensioned structural anchors have also been used by others to stabilize new structures. The Wanapum project included a hydropower station designed to provide for six future generating units. To reduce initial costs, only the intakes for these six future generating units were constructed. Since the entire structure was not built, the intakes lacked stability. The most practical and economical solution was to use tensioned structural anchors to stabilize the monoliths (Eberhardt and Veltrop 1965).

8-2. Anchoring Structures to Rock

a. Tensioned structural anchors. Inclined, tensioned structural anchors are effective in increasing the stability of structures by improving factors of safety against sliding and uplift, and improving the resultant location. The number, orientation, and capacity of anchors should be based on engineering considerations and stability requirements. The process of installing tensioned anchors consists of:

- drilling holes across the potential failure plane to accommodate anchor installation
- installing the anchors
- grouting the fixed or dead end of the anchors
- jacking at the stressing or live end of the anchor to induce an active tensile force which will provide a clamping action across the potential failure plane
- grouting the remaining ungrouted length of the anchor for corrosion protection.

Fixed end anchorage is created by grouting the annular space around the tendons for a length of tendons (bond length) sufficient to develop the ultimate tensile capacity of the anchorage system. Full active tensile load is developed in the remaining ungrouted (free stressing length) of the tendon anchor system by jacking at the free end of the tendons. The tensile force induced in the free stressing length of the anchor loads the potential failure plane with a compressive stress normal to the failure plane and for inclined anchors, a horizontal component that counteracts applied shearing loads. Allowances are made in the jacking load for anticipated losses such as relaxation of the tendons, seating losses, creep, and foundation consolidation.

b. Anchor System Selection. There are a large number of products and systems to choose from, with the primary considerations being cost and corrosion protection. Generally, fewer higher capacity anchors are less expensive than more smaller capacity anchors, however with high capacity anchors, the failure of a single anchor can result in a significant loss of sliding resistance along the potential failure plane. Single, high-strength steel bars have the smallest capacity of tensioned anchors, while steel strand is used when large capacity anchors are needed. Single bars provide less force because they have a lower working stress than strands. The three most common types of anchor systems are the hollow groutable anchors, the solid continuously threaded bar anchors, and strand anchors. Following is a brief description of each system.

(1) The hollow-groutable anchor system consists of a threaded high-strength reinforcing bar with a hole in the middle to permit the annular space around the bar to be grouted after tensioning is complete. Anchorage of the dead end is accomplished by an anchor cone, which engages the existing concrete. The bar is tensioned by hydraulic jack, or by tightening the nut located at the stressing end of the unit.

(2) The threaded-bar anchor system is tensioned with a center pull jack. A special threaded nut is used for locking- in the applied tension. The threaded bars may be secured in two-stage cement grout or fast- and slow-set polyester resin grouts. Long-term creep of polyester resin has been a concern that stems from laboratory tests at high-bond stresses. Generally, creep at normal working-level stresses will not be a problem.

(3) The strand anchor system is a seven-wire type having a center wire wrapped tightly by six outer wires. Strand anchors use single- or two-stage grouting with the single-stage grouting being referred to as an unbonded anchor, while the two-stage grouting procedure is referred to as a bonded anchor. The free stressing length of bonded anchors are grouted after the anchor has been stressed.

c. Untensioned structural anchors. Untensioned anchors shall not be used to improve sliding stability or resultant location since they may require excessive displacement before becoming effective. However, untensioned structural anchors may be used for the following situations.

(1) Walls. Anchored walls should be considered in areas where the rock is strong and firmly in place but is subject to surface expansion and exfoliation upon exposure. The concrete wall acts as a lining, which serves to limit the surface deterioration and to protect it against erosion from flowing water. The lining should be anchored adequately by untensioned structural anchors grouted into drilled holes in the rock. Unless the rock mass is essentially free of jointing, the anchorage should extend deep enough to engage a sufficient mass of rock for stability with a factor of safety where the anchored portion is assumed to be separated from the adjacent rock by continuous cracks. In some cases, the existence of definite slip planes dipping into the channel may require deeper anchorage near the top of the wall. Anchors may be installed at any inclination, except inclinations less than 5 degrees from horizontal, which should be avoided due to difficulties with installation. Horizontal and upwards-sloping anchors require specialized grouting techniques. Drainage should be provided to reduce the hydrostatic pressure behind the wall as much as practicable. Drain holes drilled into rock with outlets through the concrete generally provide the best and simplest system. The drain holes are sloped down a little toward the outlet and should be deep enough to adequately drain those cracks within that rock mass necessary for stability, plus the area immediately behind the mass. In cold climates, concrete thickness required for frost protection of poor rock may determine the wall thickness. Otherwise, the anchorage and concrete lining are designed to withstand the reduced hydrostatic pressure.

(2) Slabs. Anchored slabs on rock are designed to withstand uplift and other probable forces with an ample factor of safety. If the slab is on suitable rock, the slab can be anchored with untensioned structural anchors grouted in holes drilled into the rock. The holes shall have a sufficient depth to engage a mass of rock the submerged weight of which will withstand the net upward forces, assuming the mass of rock bounded by a 90-deg apex angle at the bottom with allowance for overlap from adjacent anchors. Drain holes drilled into the rock and discharging through the slab can reduce unbalanced hydrostatic uplift. Depth and spacing of drain holes should be determined by consultation with the geotechnical engineer. The holes should be inclined at a small angle from normal to the slab so that their outlet end is downstream from their inlet end to avoid a possible increased uplift from flowing water. A pervious drainage blanket under the slab with transverse perforated drain pipes discharging through the slab may be economical where a blanket is needed to protect the foundation from freezing.

8-3. Tensioned Anchor Loads

For inclined structural anchors, the normal component of the anchor force adds to the frictional resistance by increasing the normal force on the sliding plane, and the tangential component acts against the driving forces as resisting forces. The horizontal component should be treated as an applied load (see example D5 in Appendix D). Adding the horizontal component to the frictional resistance is not logical since the frictional resistance is a function of the normal loading and angle of internal friction, while the tangential component is related only to tendon load and inclination of the tendon. These applied loads occur immediately upon stressing the anchors and are not associated with deformations accompanying the development of frictional resistance. Thus, it should be considered an applied load that induces shear stresses along potential sliding planes opposite in direction to those due to downstream forces on the structure.

Rehabilitation of existing structures should examine a range of anchor forces varying from the force needed to meet Chapter 3 stability requirements to minimal forces needed to meet the requirements discussed in Chapter 6. A check should be made to ensure that the tangential component is never so large as to cause upstream sliding. An example might be that the tangential component applied to a dam is equal to or larger than the reservoir load on a dam. This raises the question of possible upstream sliding should the reservoir ever be evacuated.

8-4. Structural Anchor Design

a. Tensioned structural anchors. Recommendations for Prestressed Rock and Soil Anchors (Post-Tensioning Institute 1996) provides guidance in the application of permanent and temporary utilizing high-strength prestressing steel (includes bars and strand guidance). The recommendations do not deal with the design of anchored structures in general, but are limited to considerations specific to the prestressed anchors. Design, materials, fabrication and handling, installation, and performance testing of permanent tensioned structural anchors should be in accordance with PTI 1996 subject to the following provisions.

- PTI Chapter 3.0, "Specifications and Responsibilities" is not applicable because the structural anchors are a part of a larger Corps project and, as such, the structural anchors are included as a part of the contract documents.
- PTI Chapter 4.0, "Materials" requirements shall be used for prestressing steel (strand, bar, indented strand, epoxy-coated strand, epoxy-coated bar), anchorages, couplers, centralizers and spacers, corrosion inhibiting compound, bond breaker, sheath, tendon bond-length encapsulations, heat-shrinkable sleeves, grout tubes, anchor grout, cement grout, and polyester resin grouts.
- PTI Chapter 5.0, "Corrosion Protection," provides guidance for two classes of corrosion protection. Class I, encapsulated tendons, is often referred to as double corrosion protection while Class II, grout protected tendons, is often referred to as single corrosion protection. Class I corrosion protection shall be used for the anchors.
- PTI Paragraph 6.4.2 discusses fully bonded and unbonded anchors. Fully bonded anchors are required and shall be grouted in two stages, i.e., first stage (primary) is to grout the bond zone, and the second stage is to grout the free length after the anchor has been stressed.
- PTI Paragraph 7.4 discusses waterproofing rock-anchor drill holes. Provisions in this paragraph are a requirement.

b. Untensioned structural anchors. There is no industry standard for the design of untensioned structural anchors. Such anchors shall be ASTM grade 60 bars designed in accordance with EM 1110-2-2104. Anchors are usually of large diameter to reduce drilling cost. With large anchors however, the strain penetration that occurs in the reinforcement on each side of the failure plane is large, requiring significant elongation to develop the ultimate capacity of the anchor.

c. Bond length for tensioned and untensioned structural anchors. The required anchorage length is based upon tendon load, diameter of the tendon hole, type of tendon system, and the bond strength between the surrounding material and grout and between the tendon and grout. The bond lengths calculated below should be verified by performing pull-out tests utilizing tendon systems and drill-hole sizes matching those used by the contractor. If the foundation is variable, pull-out tests should be performed in areas representative of all site conditions and bond lengths adjusted accordingly. The embedment, and/or hooks, in the concrete for untensioned structural anchors should be in accordance with the requirements of EM 1110-2-2104. Embedment length should not be confused with the anchorage bond length of the tendon. Bond lengths for tensioned and untensioned structural anchors can be determined by using the largest value resulting from the computations in paragraphs (1), (2), or (3) below.

(1) Bond between tendon and grout

$$L = P / (\varphi \pi \ d_b f_{bu})$$

where: L = bond length

P = ultimate load for untensioned anchors and design load for tensioned anchors

d_b = diameter of tendon

f_{bu} = ultimate bond strength between tendon and grout (For design, use f_{bu} equal to $6\sqrt{f_c'}$ where f_c' is the compressive strength of grout; f_{bu} must be verified by tests as specified in PTI 1996)

φ = strength reduction factor = 0.9 (Not to be confused with soil strength parameter.)

(2) Bond between grout and rock

$$L = P / (\varphi \pi \ d_h f_{cu})$$

Where: d_h = diameter of drilled hole

f_{cu} = ultimate bond strength between grout and rock, or c, whichever is less (see Table 6.1 of PTI 1996 for values of f_{cu})

c = cross bed shear value in rock in the direction of pull out

(3) Rock-mass shear failure

(a) Tensioned structural anchors. With all tensioned structural-anchor systems, a major consideration is determining how deep to install the anchors. An anchor system that is too shallow may cause tension and cracking to occur along potential failure planes in the foundation, and a system too deep is uneconomical. PTI recommends normal bond length not less than 3.0m (10ft) for bars and 4.5m (15ft) for strand. Bond lengths greater than 10m (35ft) are normally not used. PTI recommends free stressing lengths to be at least 3.0m (10ft) for bar tendons and 4.5m (15ft) for strand tendons. Center-to-center spacing between anchors shall be at least 1.5m (5ft) unless unusual circumstances dictate. The fixed end (dead end) anchorages should be staggered.

(b) Untensioned structural anchors. For untensioned structural anchors, rock-mass failure will not normally govern design. Where the foundation consists of weak or cracked rock, a rational analysis should be performed to ensure adequate embedment length. This analysis should be performed in accordance with the requirements given in EM 1110-2-2400.

d. Anchorages for tensioned structural anchors. Anchorages shall be a combination of either a steel bearing plate and wedges, or a steel-bearing plate with a threaded anchor nut. The anchorage components must be carefully designed, detailed and constructed to provide adequate corrosion protection. It is also important to properly detail the region under the anchorage. Large tendons will exert very large forces on the bearing plate, which will then be distributed into the concrete. This will result in localized stress concentrations in the concrete at the edges and/or immediately beneath the bearing plate. Anchor details and bearing plate requirements are available from the post-tensioning system manufacturer. Guide specifications for post-tensioning materials are available from the Post Tensioning Institute (PTI), and contain information on permissible compressive concrete stresses beneath stressing-end anchor plates.

8-5. Stressing, Load Testing, and Acceptance

Field tests shall be performed before and during the installation to verify the adequacy of the anchor system and installation procedures. Tests before installation shall be used to check the performance of selected drilling method, conformability of hole size and drift tolerances, adequacy of assumed bond strengths between grout and rock, grout and concrete, and grout and tendon. Tests during installation should be adequate to ensure that anchors are installed in accordance with the requirements of the plans and specifications and that the ultimate capacity of the tendon can be developed. The number of tests required depends on the site-specific information including drilling conditions, type and/or size of tendons, and complexity of foundation formation and material. All testing procedures and acceptance criteria shall be in accordance with PTI 1996.

8-6. Monitoring Structural Anchor Performance

Grouting, which is required to protect tensioned anchors against corrosion, results in a fully bonded anchor. Untensioned anchors are installed with a single grouting process, which also results in a fully bonded anchor. Therefore, the long-term monitoring of anchor systems is limited. It is assumed that fully bonded anchors are maintenance free because the alkalies in the grout provide extra corrosion protection, the entire anchor is not lost if a portion of the anchor fails, and small, localized movements can mobilize reserve capacity in the anchor. Inspection of the structure and monitoring of its behavior are the only practical ways of monitoring performance of fully bonded, structural anchors. Provisions are being developed that will allow measurements of galvanic action within the anchor system. Also, sensors can now be embedded in fiber-optic cables. These cables have been epoxied to ship hulls, bridge girders and reinforcing bars with the sensors transmitting data in real time. It is believed they could be epoxied to anchors. These procedures may be investigated by the designers if there is a need for further assurance that an anchor system remains functional.

8-7 Mandatory Requirements

a. It is mandatory to obtain approval from CECW-E prior to use of soil anchors to meet requirements for stability of permanent structures.

b. It is mandatory to obtain approval from CECW-E prior to use of rock anchors to meet requirements for stability of new mass concrete structures.